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Evaluation, Design, and Construction of Levees

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INTRODUCTION

0.1 Purpose. The intent of this manual is to provide guidance for use by U.S. Army Corps of Engineers (USACE) for evaluation, design, and construction activities when USACE has those responsibilities. This manual can also be used as a reference document by others.

0.2 Applicability. This manual applies to all applicable to all Headquarters U.S. Army Corps of Engineers (USACE) elements, Divisions, Districts, laboratories, and field operating activities having responsibility for the planning, design, evaluation, construction, and maintenance for civil works projects.

0.3 Distribution Statement. This manual is draft and has not been finalized.

0.4 References. References are listed in Appendix A.

0.5 Records Management (Recordkeeping) Requirements. The records management requirement for all record numbers, associated forms, and reports required by this regulation is addressed in the Army's Records Retention Schedule – Army (RRS-A). Detailed information for all related record numbers is located in the Army Records Information Management System (ARIMS)/RRS-A at https://www.arims.army.mil. If any record numbers, forms, and reports are not current, addressed, and/or published correctly in ARIMS/RRS-A, see Department of the Army Pamphlet 25-403, Guide to Recordkeeping in the Army.

0.6 Scope of the Manual. This manual is for the evaluation, design, and construction of levees. Levees may be comprised of features such as embankments, floodwalls, pipes and associated drainage features, closures, pumping stations, floodways, and designed channels that are collectively integral to excluding flood water from the leveed area.

0.7 How to Use this Manual. Guidance presented in the manual primarily addresses the evaluation, design, and construction of levees comprised of earthen embankments. Guidance in the manual should be followed for other levee features (such as floodwalls, closure structures, interior drainage systems, seepage control systems, etc.) where specified to ensure the features of a levee perform collectively as a system. For levee features other than embankments, other feature specific guidance, as referenced throughout, should be followed in addition to this manual for evaluation, design, and construction of those features.

0.7.1 This manual does not provide guidance on how to determine a levee's height for purposes of excluding a desired range of flood events. This is performed during the project formulation phase (e.g., planning) which is covered in ER 1105-2-100, Planning Guidance Notebook, USACE 2000).

0.7.2 This manual does not specifically address every possible levee situation and in that regard may be considered general in nature. The manual is not intended to replace the judgment or critical thinking of the analysts and/or designers, evaluators, and constructors on a particular project.

0.7.3 This manual applies to all levees regardless of loading duration (e.g., intermittently loaded or continuously loaded) including both riverine levees and coastal levees.

0.8 General Requirements. General requirements are provided below for the evaluation, design, and construction of levees associated with USACE civil works projects.

0.8.1 Evaluation of Existing Levees. Levee evaluations may be performed to determine an existing levee's performance for a range of flood events and are typically performed in conjunction with a risk assessment. This manual can be used to evaluate an existing levee's performance for a range of flood events. However, existing levees are not required to meet design and construction standards presented in this manual. For example, there are existing levees that do not meet current design and construction standards that still perform satisfactorily while there are levees that do meet current standards that do not perform satisfactorily. Traditional engineering standards, such as factors of safety, were generally developed through direct field observations to address the most common failure modes, typically stability and seepage. Risk assessments provide a framework to take into consideration past performance, site specific considerations, or uncertainties that are not fully accounted for in traditional engineering standards allowing decision makers to decide the need to modify existing levees.

0.8.1.1 National Flood Insurance Program (NFIP). This manual is referenced in 44 CFR 65.10(b)(4) as an alternative analysis, specifically Case IV (e.g., levee slope stability for steady state seepage conditions during flooding) within the 1978 version of this manual, that may be used to demonstrate the levee is designed and constructed for stability against loading conditions. This Case IV analysis is now Case III found in Chapter 7 of this manual. The manual now has analysis for different scenarios and loading conditions that is more adaptable to site specific situations. The analyses methods in this manual can still be used to evaluate levees for the purposes of the NFIP but is not required by 44 CFR 65.10. Practitioners should refer to the most current FEMA guidance for the NFIP.

0.8.2 Levee Design. Levee design is performed for new levees and modification, repair or rehabilitation of existing levees. 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 at a minimum. Resiliency of the levee during an overtopping event is also an important consideration during design. A levee design should also be economically feasible and constructable.

0.8.2.1 Design of a levee is often an iterative process that requires initial evaluation and design as well as adjustments during final evaluation and design. Forward advancements and experience in risk assessments as an evaluation tool now provides practitioners a more deliberate process to better supplement the levee evaluation and design. Risk assessments provide a framework to take into consideration past performance, site specific considerations, or uncertainties that are not fully accounted for in traditional engineering standards allowing designers or adjust a traditional design to ensure the levee provides the intended flood risk reduction. Refer to Chapter 1 for levee design procedures.

0.8.3 Construction. Levee construction occurs when there is physical construction of a new levee feature or existing levee feature that is being modified, repaired or rehabilitated. The general requirement for levee construction is to construct the levee as intended to achieve the intended flood risk reduction benefits of the levee project. The levee project should also be constructed in a cost effective and timely manner, in a manner that minimizes and reduces impacts to environmental, cultural, and natural resources.

0.9 Terminology.

0.9.1 Levee. A levee is a man-made barrier along a watercourse with the principal function of excluding flood waters from a limited range of flood events from a portion of the floodplain (referred to as "leveed area").

0.9.2 Levee Embankment. An earthen embankment is the most typical feature associated with a levee, and for many levees it can be the primary (or even only) physical feature. However, they often work in concert with other features which support the function of excluding floodwaters. Basic components of an embankment feature of a levee are shown in Figure 0-1. Additional components of an embankment are sometimes required to ensure the reliability of the embankment such as seepage control measures (Chapter 6), stability berms (Chapter 7), and erosion protection (Chapter 9).

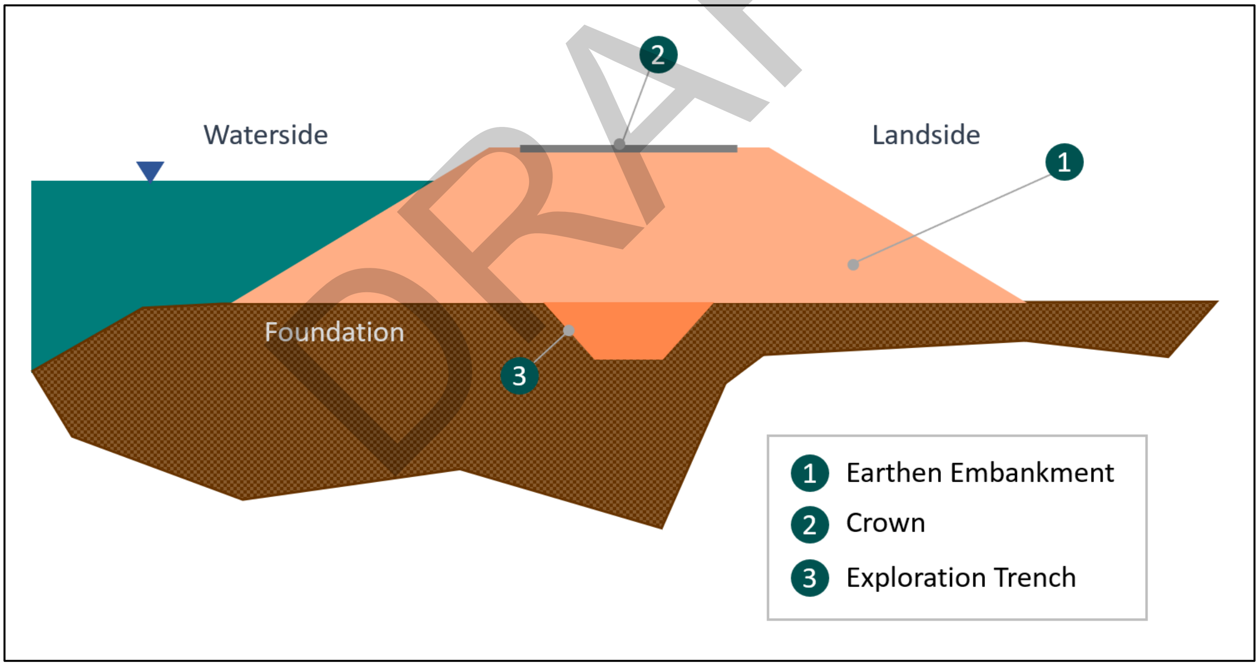
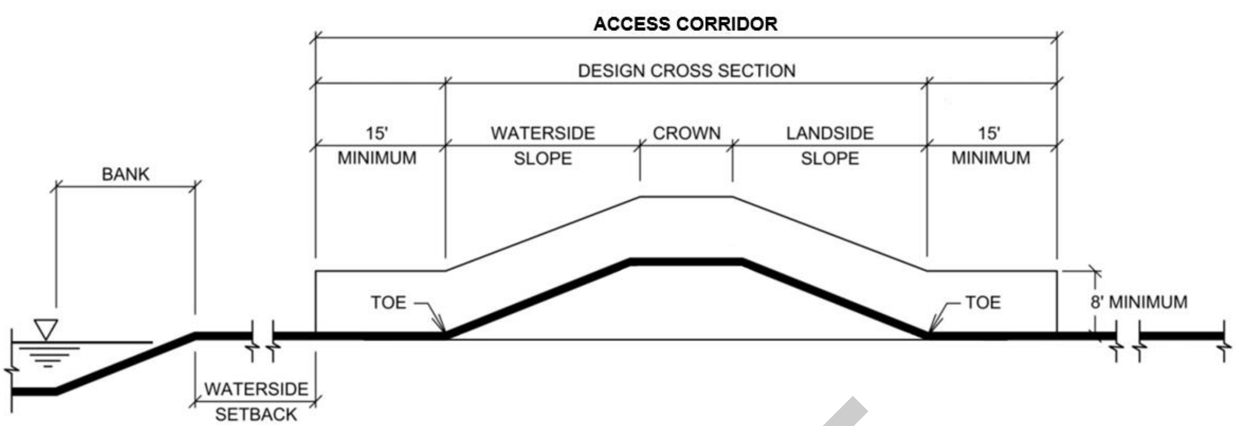
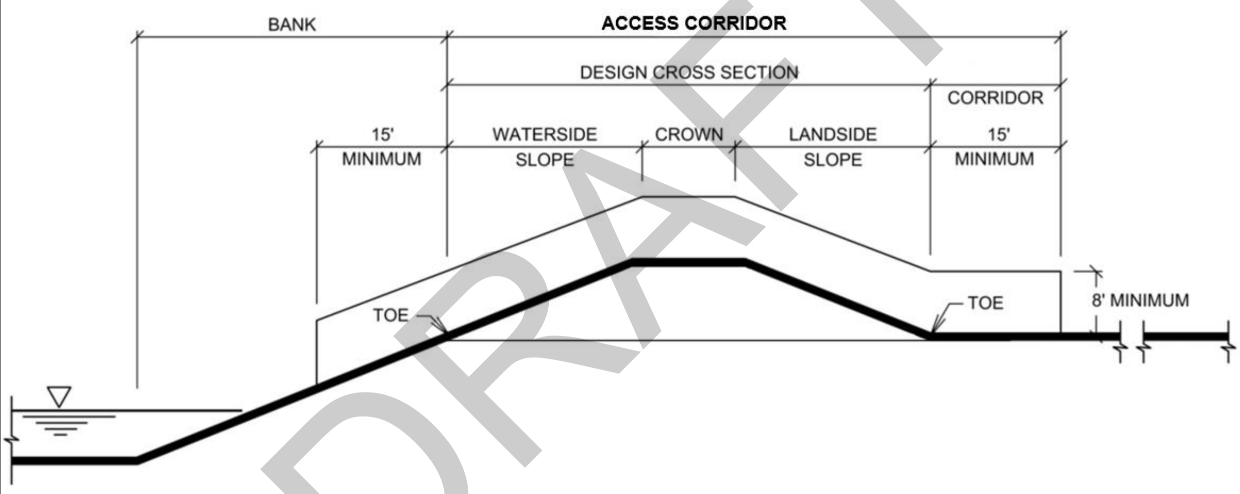


Figure 0-1. Basic Embankment Component

0.9.2.1 Access Corridor. An access corridor is a defined area that in includes the levee and a certain distance beyond the toe on each the waterside and landside. This is an area that is maintained to always have dedicated access on and near the levee to conduct inspection, operations, and maintenance activities, including emergency operations and repairs. Access corridors, as illustrated in Figure 0-2, are critical to ensuring long term levee reliability.



a. Basic Levee (above).



b. Bank Levee (above).

The purpose of these illustrations is to define terms preferred conditions. These illustrations do not include appurtenant structures and do not represent all possible levee configurations.

Figure 0-2. Typical Levee Cross Sections

0.10 Manual Organization. Chapter 1 provides guidance on how to perform levee evaluation and design and should be the starting point for any user of the manual to perform these activities. The remaining chapters and appendices provide guidance for key components of levee evaluation and design as well as levee construction and transition to operations and maintenance.

CHAPTER 1

Levee Evaluation, Design, and Construction Procedures

1.1 Purpose. The chapter provides the basis for all levee evaluations, designs, and construction including the procedures that should be followed when performing these activities.

1.2 Levee Evaluation and Design Process.

1.2.1 General. The levee evaluation and design process involve an initial deterministic evaluation and design (referred to as Phase 1) followed by an evaluation of the levee project with a risk assessment to inform any necessary design adjustments to finalize the evaluation and design (referred to as Phase 2) as described in Section 1.2.3. Construction of the levee (refer to Chapter 10) typically occurs after the final evaluation and design is complete.

1.2.1.1 The design of new levees, modifications, or rehabilitation of existing levees will use a risk assessment to supplement the design process to better evaluate potential performance and cost effectiveness. Risk assessments can enhance the evaluation and design process by -

(1) Taking to account local site-specific conditions, such as climate or loading conditions, that are not accounted for in deterministic standards that are developed from empirical observations for a limited range of conditions.

(2) Highlight critical potential failure modes that could be unique to a specific levee.

(3) Explicitly account for uncertainty in the design parameters and methods leading to uncertain levee performance.

(4) Account for planned flood fighting and human intervention related to successful levee performance.

1.2.2 The levee evaluation and design process are discussed in the following subsections.

1.2.2.1 Phase 1. Perform initial deterministic evaluation or design. The initial deterministic evaluation (e.g., existing levee) and design (i.e., new levee or modification to an existing levee) follows the usual design 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 (see Chapter 4). Thus, consider potential consequences in the leveed areas and determine the need for the evaluation or design to have greater confidence and reliability by reducing uncertainty through more comprehensive investigation and analyses.

1.2.2.2 Phase 2. Evaluate and adjust design as necessary using a risk assessment. The initial deterministic design should be adjusted when necessary, according to the results of the risk assessment performed on the initial deterministic design. When evaluating existing levees, the risk assessment should be used to inform any necessary adjustments (i.e., modifications). In higher risk situations, such adjustments may include the addition of complementary resilience measures to increase robustness, redundancy and recoverability. In lower risk situations, a value engineering approach may be adopted in order to remove costly design features that are not critical to the levee's performance. It should also be acknowledged when making these decisions that risk may change over time and those potential changes should be factored into the decisions to adjust designs. The final evaluation and design are completed in this phase. Adjustments may not be necessary for all levee designs. Considerations on when adjustments should be made are provided below.

1.2.2.3 Adjustments to the initial deterministic design should only occur to:

(1) Adequately address risk-driving potential failure modes that exist from the initial deterministic design;

(2) Ensure the levee will perform adequately for a full range of loadings. To the extent possible, ensure that the levee will not breach before it is overtopped;

(3) Incorporate additional features that could make the levee more resilient without significant additional cost; or

(4) To reduce levee risk to 'tolerable' levels. Refer to the most current USACE guidance on how to determine 'tolerable' levee risk for USACE projects.

1.2.2.4 Then the next step would be to assess the flood risk and levee risk with the adjusted design and repeat the evaluation as needed to optimize cost, design goals, and flood risk reduction objectives.

1.3 <u>General Procedures.</u> The general process with the corresponding chapter in the manual is shown in Table 1-1, but level of effort and rigor will vary from project to project.

Table 1-1. General Procedures for Evaluation, Design, and/or Construction of Levees Systems

Step	Procedure	Chapter
1	Develop a preliminary geotechnical and geological characterization of the levee system based on a thorough review of available data including analysis of aerial photographs, assessment of hydrologic conditions, compilation of available subsurface data and, for existing levees, compilation of performance data. For new levees, preliminary subsurface explorations may also be initiated.	Ch. 2
2	Select appropriate features and levee cross-section(s) for performing analyses and conduct a Potential Failure Mode Analysis (PFMA) based on a thorough review of available data including design and construction reports, construction photographs, previous inspection and risk assessment results, emergency action reports, flood fighting reports, and discussions with local sponsors and on-site operation and maintenance personnel. The principal causes of levee failure are (or are combinations of): 1. Overtopping 2. Underseepage, through-seepage, and internal erosion 3. Surface erosion 4. Slides within the levee embankment or the foundation soils 5. Collapse or seepage along conduits, culverts, and pipes	Ch. 1 Ch. 5
3	Obtain additional information if existing information is inadequate, including, but not limited to: a. Additional information on soil profiles b. Strengths of foundation and levee materials c. Detailed information to establish reasonable models to understand pore-water pressures that currently exist and forecast pore-water pressure and seepage quantity during flooding d. More detailed information on borrow areas and other required excavations	Ch. 2 Ch. 3
4	Update the geotechnical and geological site characterization and divide the entire levee into reaches that have similar foundation conditions, features, heights, fill material, etc. and complete the following within each reach: a. Assign at least one typical trial analysis cross-section for analysis in each reach. In some circumstances, more than one trial cross-section within a reach may be required. b. Use all available information to verify or assess both embankment and foundation analysis parameters (e.g., soil mechanics parameters to support “deterministic” analyses for comparison to current levee design criteria; distribution properties of soil mechanics parameters to support “probabilistic” analyses for levee reliability). c. For design and construction projects, compute rough quantities of suitable material and refine borrow area locations.	Ch. 4 Ch. 5

Step	Procedure	Chapter
5	Complete an initial design for each trial cross-section to comply with deterministic design criteria (e.g., slope stability factor of safety for flood loading equal to or greater than 1.4). Analyze each trial cross-section as appropriate for: a. Levee height b. Underseepage and through-seepage c. Slope stability d. Settlement e. Erosion f. Trafficability of the levee surface Hydraulic load conditions used for analysis will include all potential loadings up to and above the top of levee. For levee features not covered in this manual, refer to other EMs specific to those features for deterministic design criteria.	Ch. 1 Ch. 6 Ch. 7 Ch. 8 Ch. 9 Ch. 11 Ch. 12
6	Conduct a risk assessment using current methods to evaluate the reliability of each trial cross-section that was initially designed to meet deterministic design criteria. a. Proceed to the next step if the levee reliability of the initial design efficiently meets project objectives. b. Revise the design and re-iterate through the design process when the levee reliability of the initial design does not efficiently meet project objectives. This may require upscaling the design for factors of safety higher than the deterministic design criteria in this manual when higher levels of levee reliability are required. Alternatively, this may require downscaling the design to meet factors of safety lower than the deterministic design criteria. Deviations from deterministic design criteria will need to for the USACE approval process.	Ch. 1
7	Finalized design for construction including: a. Compute final quantities needed, evaluate and select final borrow area locations. b. Prepare plans and specifications for construction. c. Develop a draft operations and maintenance manual. d. Prepare pre-construction design documentation report.	Ch. 4 Ch. 10 Ch. 12

Step	Procedure	Chapter
8	Construct levee project with the following: a. Proper preparation and treatment of the levee foundation and the construction of the levee embankment such as exploration trenches and fill selection, fill placement, and fill compaction. b. Design considerations to improve levee embankment stability during construction. c. Sequence and coordination of construction activities. d. Construction quality control and quality assurance requirements for levee projects. e. Post construction documentation (including as built drawings, final design documentation report, and final operation and maintenance manual). f. Post construction risk assessments required for levee projects including update to the National Levee Database.	Ch. 10 Ch. 12

1.4 Loading Conditions for Evaluation and Design. Load cases for levees generally include static loads from hydraulic, structural, and/or soil forces. Dynamic loads from navigation vessel impacts, vehicular traffic, and/or wind and hydraulic forces may be considered on a case-by-case basis depending on the levee requirements and loading conditions. The following sections define the various static hydraulic loading conditions and associated water surface elevations as illustrated in Figure 1-1. These hydraulic conditions and water surface elevations are important considerations for levee evaluation and design. However, for any levee evaluation and design, it important to assess the levee performance for all possible hydraulic conditions and water surface elevations to ensure the levee project's intended flood risk reduction is achieved. For the range of water surfaces that are assessed, it is also important to understand the annual chance of exceedance of each water level as well as the duration and the warning time of occurrence.

1.4.1 Design Water Surface Elevation. The design water surface elevation (DWSE) corresponds to the congressionally authorized water loading for the project. Levees designed and built for water loading to this elevation would be required to have a very small probability of failure. Uncertainty is typically explicitly addressed in the USACE planning process, resulting in DWSE as shown in Figure 1-1. All water loadings up to the DWSE are expected to result in very low likelihood of unsatisfactory performance that could lead to breach.

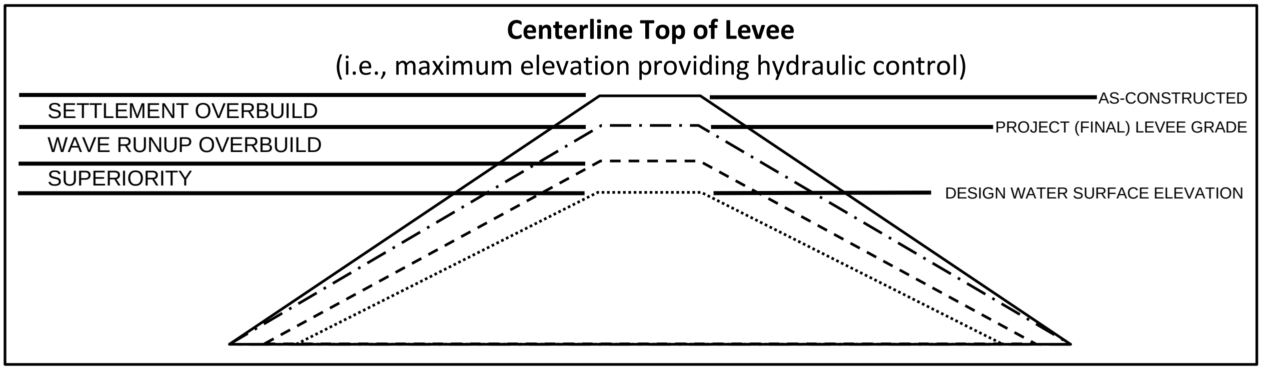


Figure 1-1. Levee section components and top of levee nomenclature. Settlement overbuild refers to additional levee height required to account for expected settlement. Wave runup overbuild refers to additional levee height required to limit wave over-wash. Superiority refers to additional levee height placed at selected locations to induce overtopping at other predefined locations.

1.4.1.1 The DWSE considers geotechnical, construction, hydrologic, and hydraulic uncertainties and is generally based on optimizing net project benefits (i.e., maximized National Economic Development (NED), ER 1105-2-100). Loss of life and other factors are also considered during the USACE planning process for determining the DWSE. The DWSE is the water level for which all initial deterministic design factors of safety (i.e., initial deterministic design) will apply.

1.4.1.2 The DWSE will be used in all deterministic design analyses to determine configurations and features necessary to meet initial deterministic design criteria.

1.4.2 Top of Levee (As-Constructed and Final Levee Grade). Deterministic analyses presented in this manual should be performed for flood loading conditions at the top of levee to evaluate the levee performance under this loading condition. The top of levee will be the as-constructed top of levee grade for new levees and project (final) grade for existing levees. There are no deterministic criteria for these flood loading conditions. Results from these analyses should be used to inform any design adjustments with a risk assessment (e.g, Phase 2).

1.4.3 Overtopping and Other Load Cases. There may be other water levels of interest that designers should analyze for consideration during final evaluation and design. This may potentially include different combinations of flood loading at lower levels, consideration of water surfaces above the top of levee during overtopping, and a range of landside tailwater and ponding conditions.

1.4.3.1 Levee raises, such as using sandbags, that may be implemented during an emergency response should be evaluated to determine how those measures may impact the levee reliability if not during design, at least prior to any implementation. This is especially important if superiority is factor in establishing the top of levee.

1.5 Key Levee Designations.

1.5.1 Project (Final) Levee Grade. As shown in Figure 1-1, the top of barrier associated with the DWSE plus superiority plus over-wash height (or wave runup overbuild) is termed the project (final) levee grade top of levee. Where superiority and over-wash height are zero, the final levee grade equals the DWSE. Levees designed in conjunction with riverine projects may include superiority (e.g., extra levee height in selected locations to ensure overtopping at other predefined locations within the levee system). As an example, if the desired overtopping location within the levee is at the downstream end, superiority overbuild may be zero at the downstream end of the levee and 1 foot at the upstream end to induce any overtopping at the desired location. Levees adjacent to open bodies of water and vulnerable to wind wave action may have an increased top of levee elevation in excess of the DWSE, called wave runup overbuild, to keep wave over-wash (volume of water from wave runup that reaches the dry side) below some acceptable limit.

1.5.2 As-Constructed (Top of Levee). The as-constructed top of levee represents the project (final) levee grade plus overbuild for settlement top of levee (as shown in 1-1). It is the highest elevation on the levee providing hydraulic control, per the original design intent. Standard consolidation and settlement analyses using physical properties of the foundation and embankment materials should be used to set the top of levee for construction to account for settlement, shrinkage, cracking, geologic subsidence, and construction tolerances as discussed in Chapter 8. Because of settlement, the elevation of the constructed top of levee is often a temporary condition and is expected to decrease with time. Typically, topsoil or road surfacing materials are placed above the selected top of levee so these materials are not relied upon as contributing to flood risk reduction (local practice may differ). However, these materials often exist and may at some point be used to contain flood loads. Seepage and slope stability analyses should include these materials for load cases analyzed.

1.6 Risk Assessments for Evaluation and Design.

1.6.1 General.

1.6.1.1 As discussed previously, levees have been designed and constructed to meet standards generally associated with a single DWSE or load level. Levee design and construction standards provided in this manual will serve as a basis for initial evaluation of the levee design and construction; however, a risk-informed approach for levee design and construction will be required to confirm the resulting levee system reliability is commensurate the potential consequences behind the levee. Through a risk-informed approach, levees will be evaluated for predicted performance for all potential water or loading levels.

1.6.1.2 Risk is defined as the combination of likelihood and consequences of flood inundation to the leveed area as shown in Figure 1-2**Error! Reference source not found.**. Flood risk associated with levees is evaluated in three components: the hazard (i.e., magnitude and frequency of loading), the predicted performance (i.e., probability of inundation for a given magnitude of loading) and the consequences (for example: lives, dollars, and other societal values lost due to flood inundation, dependent on exposure and vulnerability). Risk associated with flood inundation to the leveed area requires evaluation of the complete levee system for all potential loadings.

Figure 1-2. Components of risk evaluated for levees.

1.6.1.3 The term *flood risk* (commonly referred to as residual risk) is used to describe the risk of flooding in the area behind the levee regardless to what led to that flooding. The term *levee risk* is used to refer to the risk imposed by the levee itself. The inundation to the leveed area includes the following four *inundation scenarios* depicted in Figure 1-3: (1) levee breach prior to overtopping (the levee breaches from a defect in the levee system), (2) levee overtopping with breach (the levee overtops and breaches due to erosion), (3) malfunction or improper operation of levee system components (a component of the levee fails such as a pump or closure structure), and (4) levee overtopping without breach (also referred to as *non-breach risk*– when the floods exceed the capacity of the levee, but it does not breach). These four inundation scenarios should be evaluated to support the risk-informed decision framework for levee evaluation, design, and construction activities.

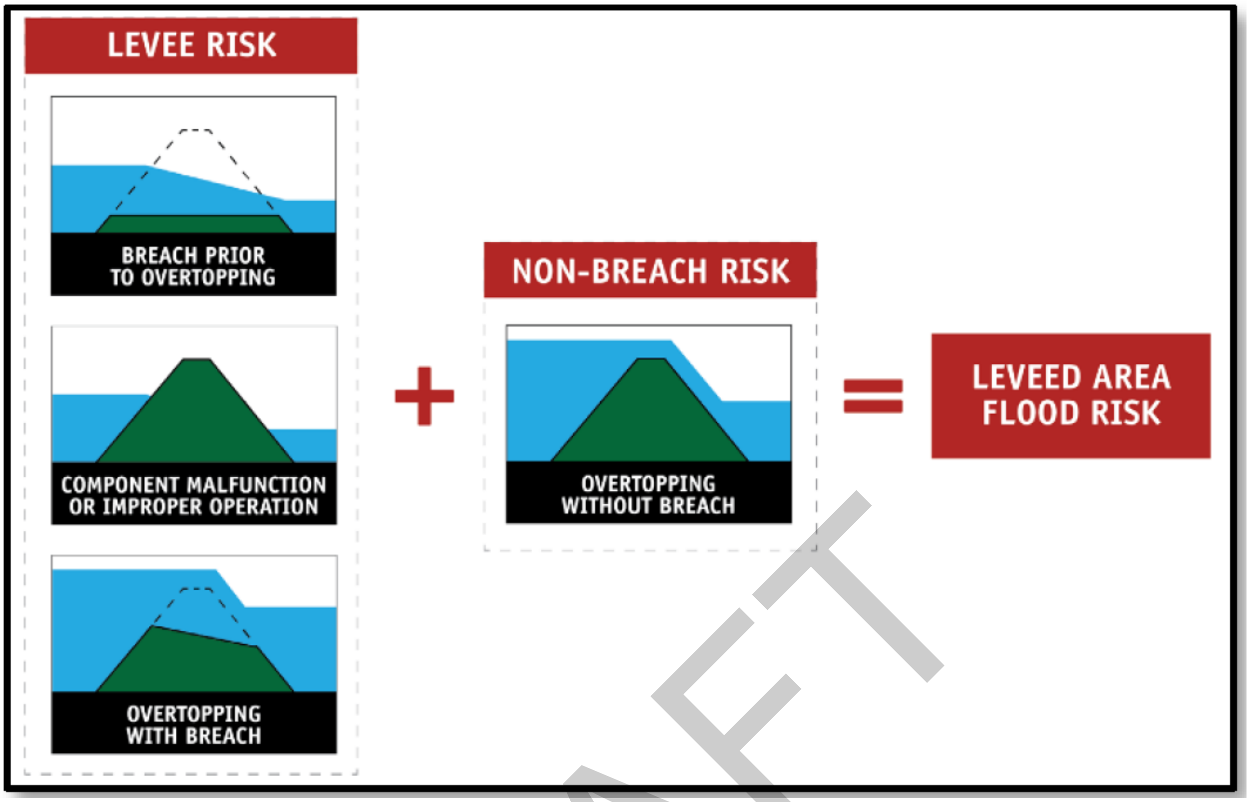


Figure 1-3. Four Leveed Area Inundation Scenarios

1.6.1.4 Levee risk should also be considered when evaluating and selecting design components, implementing construction activities, and operating the levee system. The non-breach risk is commonly evaluated to establish levee heights and other features during the USACE planning process prior to implementation of levee design and construction.

1.6.1.5 The Risk Management Center is responsible for the development, dissemination, and training of risk assessment methodology used by USACE for dams and levees. The Best Practices in Dam and Levee Risk Analysis (USBR and USACE 2019 or most recent) serves as the USACE basis for the overall philosophy, methods, and approach to risk assessments for levee systems.

1.7 <u>Risk Assessment Procedures</u>.

1.7.1 A risk assessment consists of evaluating the annual probability of inundation (also referred to as annual exceedance probability) for all levee potential failure modes for all system components (including “human” systems that require operation such as closure structures). In the risk assessment, the frequency of loading is combined with the conditional probability of failure for a particular level of loading, to yield estimates of annual probability of inundation for that level of loading.

1.7.2 The procedures for performing a risk assessment for design consists of the following steps:

(1) Hazard Assessment. Identify the potential hazards (sources of harm) to be considered. For levee systems, the typical hazards are flood, seismic, and security (intrusions, attacks, or effects of natural or manmade disasters).

(2) Levee Reliability Analysis. Assess all credible and significant potential failure modes for the levee design considering levee breach prior to overtopping, malfunction of levee system components, and levee overtopping with breach inundation scenarios associated with all loading levels from the hazard assessment ((1) above).

(3) Levee Reliability Estimate. Estimate the annualized exceedance probability for each credible potential failure modes for all potential loadings.

(4) Consequence Assessment. Estimate potential life loss and economic damages for each inundation scenario in (2) above. Consider any other potential consequences such as cultural resources, that the risk assessment may not have been able to quantify.

(5) Assess the life safety and economic benefits that would be achieved by incorporating a levee design component or taking an action, measured by differences between before-action and after-action estimated flood risks.

1.7.3 Appendix B (Quantitative Fragility/Reliability Analysis Example) of this EM provides an example of implementing the levee design process outlined in Section 1.2.

1.7.4 Practices for to evaluation hazards (hydrologic, seismic, and security) and consequences (e.g. economic, environmental, and life safety) are beyond the scope of this manual. The publications below should be consulted for evaluation of hydrologic hazards. Chapter 7 (Slope Design) of this manual should be consulted for seismic hazards. Best Practices for Dam and Levee Safety Risk Analysis should be consulted on evaluation of consequences.

1.7.4.1.1 <u>Hazard Assessment.</u> The following list of references is provided to assist in the hazard assessment. The Best Practices for Dam and Levee Safety Risk Analysis should be utilized for guidance on performing a hazard assessment.

- Standard Project Flood Determinations, EM 1110-2-1411
- Hydrologic Analysis of Interior Areas, EM 1110-2-1413
- Hydrologic Frequency Analysis, EM 1110-2-1415
- River Hydraulics, EM 1110-2-1416
- Flood-Runoff Analysis, EM 1110-2-1417
- Hydrologic Engineering Requirements for Flood Damage Reduction Studies, EM 1110-2-1419

1.7.5 <u>Levee Reliability Analysis.</u>

1.7.5.1 <u>Potential Failure Modes.</u>

1.7.5.1.1 To assess levee reliability, an assessment of levee potential failure modes should be performed. A potential failure mode is a specific manner or way that a failure of some component of a system occurs, leading to undesirable consequences, such as inundation of the leveed area, property damage, and loss of life. In general, significant potential failure modes for all levee system components and features must be described fully, from initiation to breach. In general, there are three parts to the potential failure mode description:

- The Initiator. For example, this could include increases in river stage due to flooding, strong earthquake ground shaking, or malfunction of a pump station, gate, or closure.
- Failure progression. This includes the step-by-step mechanisms that would lead to the breach or uncontrolled release of water into the leveed area. The location where the failure is most likely to occur should also be highlighted. For example, this might include the path through which materials will be transported in an internal erosion situation, the location of overtopping during a flood event, or anticipated failure surfaces in a sliding situation.
- The resulting impacts. The process and expected magnitude of the breach or uncontrolled release of the river/body of water into the leveed area is also part of the description. This would include how rapid and how large the expected breach would be and the breach mechanism. For example, the process and expected magnitude of a breach from an internal erosion failure mechanism adjacent to a conduit passing through a levee may be described as progressive sloughing and unraveling of the landside slope as a result of flows undercutting and eroding the landside toe of the levee, until the levee is breached at which point rapid erosion of the embankment remnant ensues, allowing an inflow of water causing inundation of the leveed area, with loss of life and property.

1.7.5.1.2 Understanding potential failure modes is helpful in evaluating potential design components to mitigate the likelihood of a potential failure mode progressing.

1.7.5.1.3 Potential failure modes are depicted using event trees to demonstrate the sequence of events and probabilities that result in an undesirable outcome. An example event tree for an internal erosion potential failure mode is shown in Figure 1-4. Each node on the event tree has an assessed probability of the chance event occurring. Nodes on the event tree are combined to estimate the conditional probability of inundation to the leveed area as a function of the load level.

1.7.5.1.4 A general list of levee potential failure modes is provided in Table 1-2. Conditions for every system are unique and designers are encouraged to brainstorm and refine potential failure modes that may be unique to their levee project. Although a brief summary of levee potential failure modes is provided here, a more detailed discussion on levee potential failure modes and potential failure mode assessments can be found in the Best Practices publication.

1.7.5.1.5 Design and construction components should also be evaluated for each significant potential failure mode compared to the baseline condition (without levee design and construction condition) and evaluated for any new potential failure modes specific to the component. The understanding of the levee potential failure modes can be beneficial to development of designs and selection of appropriate standards for design. As will be discussed later, in some situations, a deviation from those design standards may also be warranted.

Table 1-2. A General List of Potential Failure Modes List for Levees (not exhaustive)

- Internal Erosion Potential Failure Modes
    - Foundation underseepage leading to heave and/or internal erosion of foundation materials
    - Levee embankment through-seepage leading to internal erosion (or instability, see below) of levee embankment materials
    - Seepage and internal erosion around conduits (pipes) or penetrations through the levee and underlying foundation
    - Seepage and internal erosion into conduits or pipes
- Stability Potential Failure Modes
    - Levee embankment through-seepage leading to instability or unraveling of the landside slope
    - Shear failure of the levee embankment and/or foundation due to rapid drawdown of the river or stream
    - Flow slide of a poorly compacted, saturated levee embankment
    - Shear failure of the levee embankment and/or foundation due to weak soils or steep geometry in the levee embankment or foundation
    - Shear failure of the levee embankment and/or foundation due to reduction in shear strength from excess pore pressures generated during flood loading
    - Shear failure due to reduction in shear strength related to dilation (Fully Softened / Critical State Conditions) or saturation
- Surficial Erosion Potential Failure Modes (including Overtopping)
    - Erosion of the levee section due to high and/or prolonged hydraulic shear stresses induced during static or wave overtopping
    - Erosion of the levee section due to high velocity and /or turbulent channel flows
    - Erosion of the adjacent channel due to high velocity and /or turbulent flows causing progressive slope failure of the levee section
- Seismic Initiated Potential Failure Modes
    - Seismic loading leads to loss of shear strength in the levee and/or foundation leading to subsequent loss of levee crown and surficial erosion failure during post-earthquake flood loading
    - Seismic loading leads to damage of seepage control features in the levee and or foundation leading to seepage induced failure during post-earthquake flood loading
    - Seismic activity causes a fault under the levee to offset leading to damage of seepage control features and subsequently seepage induced failure during post-earthquake flood loading
- Malfunction of Levee System Components Potential Failure Modes
    - Improper operation of a levee system component
    - Structural or geotechnical failure of the levee closure system
    - Electrical or mechanical failure of pumping plant leading to inundation of the leveed area

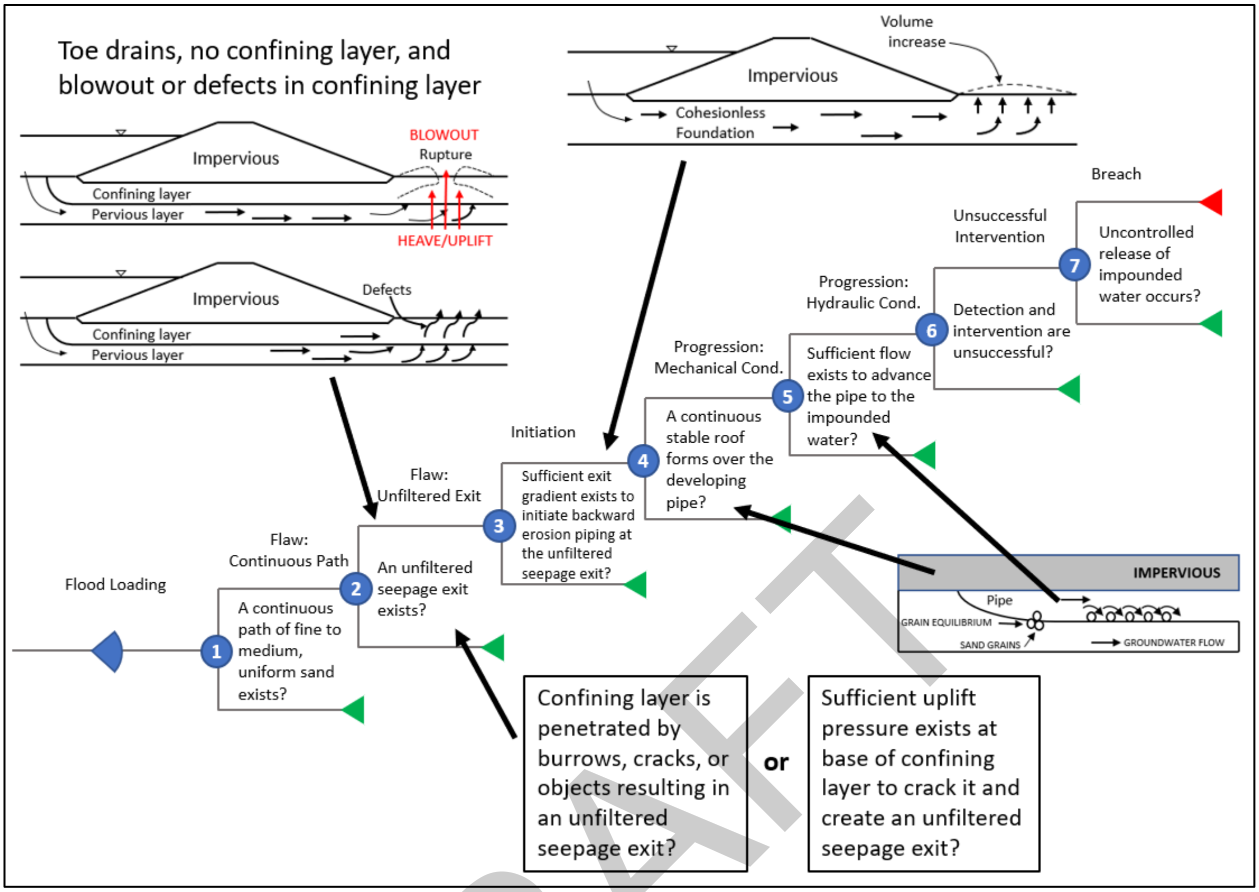


Figure 1-4. Typical Internal Erosion Potential Failure Mode Event Tree for Backward Erosion Piping in the Levee Foundation

1.7.5.1.6 Potential failure modes for levees should consider appropriate loading conditions including likelihood, magnitude, and duration of the loading. These loading conditions include during construction activities, normal operation conditions, high water events before and after consolidation of levee and foundation materials. In some settings where flood loading above the landside toe occurs relatively frequently and consequences of failure are large, seismic loading and potential poor seismic performance may also need to be considered. Normal operation conditions represent a condition that has a high likelihood of occurring (for example, 50th percentile exceedance for duration-stage) and the duration is considered to be long (weeks or longer). High water events include all potential water loadings up to the top of levee and above the top of levee (such as overtopping). High water events near the top of levee and above the top of levee are generally less likely to occur and may or may not be long in duration (less than a week) depending on the nature of the hydrologic event. Seismic loading conditions, when applicable, should consider the combined likelihood and magnitude of the seismic event and a concurrent and/or subsequent hydraulic loading on the potentially damaged levee. For many levees, the seismic event is not likely to occur while the levee is experiencing a design flood hydraulic loading; however in some settings, post-seismic flood loading may occur before post-seismic repairs can be made, sometimes warranting seismic design measures to ensure the system maintains post-seismic flood mitigation capabilities.

1.7.5.2 Levee Reliability Estimates. The conditional probability (or likelihood) of the potential failure mode causing a failure leading to breach and inundation must be estimated for all expected loadings. The conditional probability is estimated by multiplying each of the event node conditional probabilities for each load level increment. To evaluate the risk over all load level increments, the probabilities for all increments are combined yielding an estimate of the average annual probability of inundation and its complement, the average annual probability of good performance, referred to as reliability (i.e., 1-Probability of Inundation = Reliability). The process to quantitatively evaluate the potential failure mode conditional probabilities and aggregate annual probability of inundation is dependent on the complexity of the potential failure modes and is briefly described in the following paragraphs. Example calculations are provided in Appendix B.

1.7.5.2.1 Uncertainty in Levee Reliability Estimates. Estimates of probability of inundation should incorporate uncertainty and its potential impact on the assessed levee risk. The following is a brief discussion of the types of uncertainty that should be evaluated and explicitly incorporated in risk-informed design. Means to incorporate uncertainties when performance history is absent and/or when uncertainties cannot be explicitly assessed and managed in a risk-reduction efficient manner are also discussed.

1.7.5.2.2 Natural Variability (Aleatory). Some event node probabilities on the potential failure mode event tree can be characterized with physics/mechanics based numerical models that combine physical, statistical, and probabilistic methods. Physical numerical models, such as seepage, slope stability, consolidation and seismic stability models can be developed to make estimates of gradients, factors of safety, and/or displacements. Statistical methods are then employed, using estimates of the expected values and associated distributions for the parameters used in the models. This approach explicitly evaluates the impact of natural variability (typically referred to as "aleatory" in formal risk uncertainty evaluations). By propagating the aleatoric uncertainty through the model computations, the distribution of potential outcomes, often represented by a frequency distribution of a "factor of safety," can then be compared with an associated limit state differentiating poor from acceptable performance (for example, factor of safety equal to 1). Comparison of the distribution of the factors of safety and the limit state allows an estimate of the conditional probability of failure for the load condition analyzed. Simplified solution methods such as First-Order Second-Moment (FOSM) and more complex Monte Carlo simulations can be used to estimate the conditional probability of failure for each load level, yielding a "fragility" curve. Generally, the conditional probability of failure estimated using these techniques represents a lower bound on risk, because it ignores epistemic uncertainties. Epistemic uncertainties are those having do with limited knowledge about the models being used or about the parameter values of those models.

1.7.5.2.3 Unknown-Unknowns – Epistemic Uncertainty. For geotechnical engineering, epistemic uncertainty (Knowledge Uncertainty) in many situations is more important than aleatory (natural variability), because it often cannot be estimated directly. For example, consider a typical levee seepage analysis conducted to evaluate underseepage uplift. For many fluvial sites, a blanket of low hydraulic conductivity material of some thickness will overlay a sand aquifer, directly connected to the river, with hydraulic conductivity many orders of magnitude higher than the blanket. Expected values and distributions of variation in parameters such as blanket thickness, saturated unit weight of the blanket, blanket hydraulic conductivity, and sand aquifer hydraulic conductivity can all be input in a seepage model and evaluated for a particular load condition, yielding a distribution of gradients, associated factors of safety, and associated probabilities of failure. However, because subsurface investigation testing locations (such as borings and CPT probes) may be widely spaced (say greater than 1,000 feet apart), minor geologic details can readily be missed, thereby completely invalidating the results of the mathematical physics-based model (Terzaghi 1929). An "unknown-unknown," such as an undetected sand layer connected to the river beneath the levee, invalidates any of the aleatory-based modeling results, resulting in poor levee performance at loads much lower than the original model would predict. Modeling only the aleatory would significantly overestimate the reliability and underestimate the potential risk. This form of epistemic uncertainty is referred to as "model uncertainty." If the model itself is incorrect because of an unknown-unknown, better estimates of expected values and distributions will not yield better estimates of expected performance and reliability. A completely new model to predict expected behavior likely needs to be developed. Unfortunately, the need for a new model will likely not be recognized until the levee is loaded by some future flood event and poor performance is observed.

1.7.5.2.4 Development of Risk Estimates based on Expert Elicitation. Some potential failure modes are not well characterized with physical models and an expert elicitation process may be needed to develop estimates of conditional probabilities. The Best Practices manual provides a resource on how to conduct expert elicitation and a helpful list of physical models for various potential failure modes. Separation of aleatory and epistemic uncertainty, as discussed above, is often more difficult when conducting expert elicitation and some discussion about and documentation of how the experts considered each when rendering their opinions is recommended. As discussed in the next section, the reliability of the levee may not be truly understood until the levee is loaded by some future flood event and either good or poor performance is observed.

1.7.6 Application of the "Observational Method". For geotechnical potential failure modes, both for design and reliability analyses, the epistemic uncertainty "unknown-unknown" challenge is common and well-known to the profession and has been addressed through a classic inductive-reasoning approach referred to as the "Observational Method" (Peck 1969). Depending on the depositional environment and details of the design, it is recognized that an unidentified minor geologic detail has some likelihood of existing, but it cannot be economically identified at the time of design and/or possibly during construction and explicitly accounted for in the analysis model. Sometimes, these minor details will be identified and corrected during construction. In many cases, however, to identify and respond to the potential defect not identified during design and construction, the designer makes best estimates of expected performance, considers likely worst case conditions and develops plans to respond to them, develops the design and builds the structure, and then compares expected performance from analysis with observed performance of the structure under loading.

1.7.6.1.1 Performance Confirmation of Reliability. For each load level achieved with satisfactory performance, the likelihood of an "unknown-unknown" reduces. The magnitude of the epistemic uncertainty is confirmed to be small and the likelihood of failure due to epistemic uncertainty is expected to be small. To some extent, the analyst can project this trend with some confidence for load levels somewhat higher than loads experienced to date, but caution is advised. Levee performance can degrade with the passage of time. If observed performance deviates substantially and is worse than predicted, it is likely that the model used to predict performance is flawed. Additional investigations may be necessary to refine the model, re-estimate future performance and reliability, and inform design of additional measures to improve reliability. This situation can present funding challenges, such as when there are shared costs that are meant to be evaluated and paid for up front before performance observations can be used to refine and finalize design and total costs. The alternative is to avoid this situation through highly conservative design, which will make many levee projects uneconomical. Based on a half century of levee design experience, and in particular levee performance during the 2011 lower Mississippi River Flood, the observational method has been found to be the most economical and reliable means to address epistemic uncertainty in levee design and risk assessment. Future funding challenges may require more creative approaches to establish processes consistent with this limitation and the need to test a system before confirming the reliability of a design. Further, analysts and system operators must remain vigilant; levee system components can degrade over time (especially with poor operation and maintenance practices). Satisfactory performance during one loading event may not necessarily be replicated during repeated loadings. Therefore, performance monitoring and flood fighting response remain constant obligations for all levee systems.

1.7.6.1.2 Incorporating Flood Fighting into Levee Reliability Analysis. Recognizing that it is not possible to eliminate all knowledge uncertainty on levee systems, performance monitoring and flood fighting will remain important risk reduction measures to reduce epistemic uncertainty over time. All potential failure modes should consider Unsuccessful Detection and Unsuccessful Intervention (i.e., the inverse of flood fighting effectiveness) similar to that shown in Figure 1-4 for the internal erosion failure mode event tree. For some levee designs that utilize wide seepage berms with factors of safety against heave or uplift less than one at the berm toe (also referred to as "truncated seepage berms"), some amount of routine flood fighting activities, such as construction of small sandbag ring dikes around boils, will be necessary to ensure levee reliability. For flood fighting to be explicitly included as a factor reducing the potential for levee breach, the levee performance and associated routine flood fighting activities must be explicitly detailed in the levee system's Operations and Maintenance Manual (see Chapter 12 for details). If performance of a levee is deemed too poor and flood fighting activities are more than expected per the O&M manual, a risk-informed decision framework will be utilized to evaluate the needs for additional actions. If risk is high enough, a variety of structural and non-structural actions to increase levee reliability (i.e., change the levee reliability and/or decrease potential consequences) will be considered.

1.8 Evaluation and Design Considerations and Issues.

1.8.1 Important considerations for the design of levees are:

a) A levee embankment may become saturated for only a short period of time;

b) levee alignment is dictated primarily by alignment of existing riverine and coastal features, which often results in construction on poor foundations, for which there is often only limited information;

c) lower cost borrow material is generally obtained from shallow pits or from channels excavated adjacent to the riverside of the levee, which may promote seepage under the levee and produces fill material that is often heterogeneous and far from ideal; and

d) and levees are often built up over time, often with design methods and materials that would not likely be consistent with current practices.

1.8.1.1 Because of these challenges, levee systems in many locations are designed based on a combination of physics-based engineering analytical methods and response to observations of performance. In a cycle of constant improvement, levee modifications are most often implemented in locations where performance of the feature during a flood was not consistent with design expectations.

1.9 Deviation from Design and Construction Standards.

1.9.1 Deviations from design and construction standards should comply with the policy in Engineering and Construction Bulletin 2022-7, Interim Approach for Risk-Informed Designs For Dam And Levee Projects, or the most current USACE deviation guidance and procedures. Deviations from design and construction standards should include a risk assessment that demonstrates adherence to the risk-informed design process outlined in the manual.

1.10 Access Corridors. Access corridors are defined in Chapter 0. The access corridor width should be determined case-by-case, to assure adequate access for all anticipated equipment and activities including any future levee improvements. Typically, the space reserved for the access corridor should not be less than the 15 feet from each toe.

1.10.1.1 Corridor widths less than 15 feet should only be incorporated into a levee system when evaluated through a risk-informed design process and properly accounted in O&M activities including costs.

1.10.1.2 Maximizing the distance between the water source and the levee, also known as setbacks, as illustrated in Introduction Chapter, are a resilience and sustainability measure. Setbacks can improve a community's resilience by reducing its exposure to flooding and by lessening the impact of flooding should it occur. Setbacks mitigate flood hazards by providing additional waterside floodplain conveyance and habitat for floodplain vegetation that can lessen the intensity of erosive forces acting on a levee. As a sustainability measure, the waterside floodplain provides critical habitat for riverine and riparian species. Setting back levees may dovetail with conservation efforts and partnering with agencies that manage nearby wildlife management areas may provide an opportunity to leverage conservation land for real estate needed for the setback levee footprint. While setbacks may be critical to sustainability of a floodplain, a minimum distance between the water source and levee is not specifically prescribed in this manual but are an important consideration that should be addressed in a levee system plan-formulation process (see ER 1105-2-100 for more information on planning processes and terms).

1.11 Operations and Maintenance Manuals. It is important for designers to identify long-term operations and maintenance activities that are required in order to ensure the integrity of the levee. Effort and cost related to operations and maintenance should be a consideration, but should not be used as rationale for lesser design and construction requirements than what has been determined as necessary. Appropriate methods of floodfighting activities which can be used on the levee should also be identified. Designers should ensure requirements for operations, maintenance, and floodfighting are incorporated into the Operations and Maintenance Manual for the levee system. As indicated above, for flood fighting to be explicitly included as a factor reducing the potential for levee breach, the levee performance and associated routine flood fighting activities must be explicitly detailed in the levee system's Operations and Maintenance Manual (see Chapter 12 for details).

CHAPTER 2

Data Collection and Subsurface Investigations for Levees

2.1 Introduction.

2.1.1 This chapter provides guidance and best practices for data collection and subsurface investigations for levee evaluation, design, and construction. These practices are applicable any levee project regardless of levee features (e.g., embankment, floodwalls, etc.).

2.1.2 Typically for a levee project, data collection and subsurface investigation starts with a scoping effort followed by phases of data collection and subsurface investigation typically performed in the following order:

(1) Office Study - a search for available data and information (Section 2.4)

(2) Field Reconnaissance and Survey - a field reconnaissance and survey of the proposed alignment or the existing levee and visit any proposed borrow areas (Section 2.5)

(3) Data Gap Evaluation - evaluate the adequacy and quality of data that is available and to identify the types and extent of data that should be collected (Section 2.1)

(4) Subsurface Exploration and Field Testing – perform exploration and testing to collect data necessary for the project (Section 2.7)

(5) Subsurface Interpretation - integrating information and data collected to characterize subsurface conditions (Chapter 5)

2.1.3 Data collection and subsurface investigations for an existing or proposed levee can be performed in different phases of the levee evaluation and design process depending on the data and information needs of the type of project being conducted, such as feasibility, initial deterministic evaluation and design, design evaluation and adjustment using a risk assessment, and/or construction. In general, information collected in the initial phases of the project should be supportive and complimentary to data and information needs for later project phases, with an effort to optimize total expense over the life of the project and incrementally reduce the uncertainty in the levee performance.

2.1.4 This chapter is divided into two sections: Section I – Data Collection, and Section II – Subsurface Investigations. Other U.S. Army Corps of Engineers guidance for data collection and subsurface investigations such as EM 1110-2-1804, Geotechnical Investigations, should be utilized when necessary. This chapter should be used conjunction with Chapter 5, Subsurface Interpretation, of this manual. Chapter 5 provides guidelines for subsurface interpretation, which are closely related to data collection and subsurface investigations phases and should be considered concurrently when performing data collection and subsurface investigations.

Section I
Data Collection and Review

2.2 General. The data collection effort may consist of collecting available levee design and performance data; operations and maintenance information; hydrologic data; water surface elevations and groundwater; geologic and geotechnical studies; topographic and bathymetric data; and utility and encroachment types and locations. The data collected should assist in understanding the performance of the levee feature and its foundation conditions. The data should be utilized to evaluate existing conditions as well as to evaluate data gaps requiring further evaluation. Chapter 5 includes descriptions of different types of data that should be collected to perform evaluation and design of a levee project.

2.3 Scope.

2.3.1 Data collection and review usually consists of collecting data for engineering evaluation of a levee, performing an on-site (field) reconnaissance, and performing a data gap analysis. The items that should be part of data collection and review should include but not be limited to the following:

- Levee stations, levee miles, and river miles
- Survey benchmarks
- Levee and channel configurations
- Topography and bathymetry
- Channel water surface elevations and groundwater elevations
- Geomorphology/geology/soil maps/historic aerial photographs
- Levee construction history
- Levee performance history
- Existing levee improvement measures (e.g., modifications of the levee including Section 408)
- Available exploration data (e.g., standard penetration tests [SPTs], cone penetration tests [CPTs], and vane shear tests)
- Geophysical study results
- Real estate
- NEPA (cultural, environmental, HTRW)
- Historical erosion and channel migration
- Operation and maintenance information
- Studies or design documentation

2.3.2 All the data required for comprehensive evaluation of a levee may not exist from past studies and other sources. The available data should be collected and evaluated to identify the need for further data collection which will depend on the scope of a levee project.

2.4 Office Study.

2.4.1 The office study begins with a search for available information, such as topographic, soil, and geological maps, and aerial photographs; as-built documents; past performance reports; and hydrologic and hydraulic study reports. The project delivery team members should work closely together to develop an understanding of the proposed project and the depositional and erosional processes throughout the project area. The team should include engineers and geologists with knowledge in geotechnical engineering aspects of levees and fluvial geomorphologic and geologic processes. Understanding the geomorphic processes gives insight into anticipated subsurface material types and potential performance issues. The following is a list items that may be collected during the office study, if available.

- Aerial photographs and maps with levee alignment and site-specific features
- Encroachments and utilities
- Alterations documentation (i.e., Section 408 permission request submissions)
- Topography and bathymetry contour maps
- Geology and surficial geomorphology maps
- Exploration locations and logs
- Soil maps
- Geophysical study area and profiles
- Previous field reconnaissance and surveys
- Previous subsurface investigations
- As-built documents including plans and profiles
- Past performance documentation indicating areas of good performance and poor performance
- Design water surface elevation profile and other water surface elevations including frequency (i.e., annual exceedance probability) and duration
- Groundwater contour maps
- Formal and special inspections, site visits, and reconnaissance surveys reports
- Risk assessments results including levee risk management recommended actions

2.4.2 For new levee construction, pertinent information on existing construction in the area should be obtained. This includes design, construction, and performance data on utilities, highways, railroads, buildings, and hydraulic structures. For existing levees, any available information on levee construction history, performance history, existing improvements, performance monitoring, interior groundwater information, current configuration, and field observations should be collected. Available boring logs should be secured, especially logs with laboratory test results presented along with visual volumetric classifications. Federal, state, county, and local agencies and private organizations should be contacted for any available information.

2.4.3 The National Levee Database, developed by USACE, provides a comprehensive inventory of Nation's levees and should be utilized as a key data source during the office study. The inventory includes the levee's location, general condition and risk assessment information, ongoing levee risk management actions and an estimate of the number of structures and population within the leveed area. The database is dynamic with ongoing efforts to add levee data from other federal agencies, states, and tribes. The information in the database includes reports on levees, maps that include various levee features, and various integrated federal database resources such as the National Weather Service.

2.4.4 For large levee systems with a lot of available data and information, a geographic information system (GIS) may be useful to compile and organize this information. GIS can be used interactively to analyze and to generate and display resultant thematic maps and statistics to aid in making engineering decisions. A comprehensive GIS database and mapping can be useful for completing a levee project and for future reference regarding the levee. The development of a GIS database and mapping should be project-specific. The information from GIS database should integrated into the National Levee Database when appropriate. GIS analysis should not be expected to replace, but rather supplement, traditional interpretation of levee conditions using plans, profiles, and analysis cross-sections.

2.4.5 Levee and Channel Configurations. This information may include locations of the top of the levee, the landside toe, existing seepage and stability berms, existing flood walls, existing cutoff walls, existing relief wells and piezometers, levee appurtenances (e.g., closure structures, pump stations), encroachments, penetrations, ditches and invert ditches (typically within 300 feet of the levee toe but potentially further if believed important), and adjacent river or stream channel bottom profiles.

2.4.6 Geomorphology/Geology. Information may include but is not limited to aerial photographs, soils maps, and geologic maps. An understanding of the geomorphologic processes including erosional and depositional processes should be developed in collaboration among the project geotechnical engineers and geologists. This understanding provides insight into anticipated subsurface material types and performance issues. Typically, significant geologic contacts such as the interface between Holocene (less than about 10,000 to 14,000 years before present (ybp)) and Pleistocene (older than about 10,000 to 14,000 ybp) will delineate areas and subsurface regions with likely differing geotechnical characteristics. These contacts should be identified and shown on plans, profiles, and sections when possible. Several common fluvial geomorphic features, such as oxbows, overbank deposits, point bars, marshes, and crevasse splays, are typically associated with specific design and performance challenges, and should be identified and cross-correlated with performance history when possible.

2.4.6.1 Geomorphology maps should be developed using multiple sources of information and presented with an appropriate scale. For example, in the Urban Levees Evaluation (ULE) study by California Department of Water Resources (URS 2015), surficial geomorphologic maps that provide analysis of the types of information listed below were developed. These maps were found useful in evaluation of past performance, assessment of anticipated performance, and design of remedial measures.

- Aerial photographs from before urban development (black and white stereopairs taken in 1937, approximately 1:20,000 scale)
- 1911 U.S. Geological Service topographic maps
- Published surficial geologic maps
- Early and modern soil survey maps
- Other maps and documents

2.4.7 Water Surface Elevations and Groundwater. The design water surface elevation (DWSE) and any other water levels representing a range of flood loading conditions should be collected. For existing systems, there may be several past design and past flood-level water surfaces of interest including elevation for authorized or design event, top of levee, seasonal low and high water, etc. For existing levees, observed water levels may include one or more of the highest floods on record and should be cross-referenced with performance information.

2.4.8 Existing Levee Construction History. Levee construction history is important to evaluate levee composition, geometry, and methods of placement and compaction. A large number of existing levees that were built prior to the invention of modern earth compaction equipment using dredged materials from nearby rivers using suction or clamshell dredges and placed with minimum or no compaction efforts. Information on construction history may include but is not limited to plans, specifications, as-built drawings, USACE and sponsor records, and construction documents. Photographs taken during construction are especially helpful since they may provide information on both equipment used and conditions encountered.

2.4.9 Existing Levee Performance History. Levee performance history is important when assessing levee performance for different failure modes. Past performance history may include but is not limited to District records, sponsor records, satellite imagery, aerial photograph interpretation, flood fight records, past investigation reports, and inspection reports. It should be recognized that information on past performance may be limited. In many cases, past performance information may be recorded in post-flood reconnaissance reports, repair and modification documentation (e.g., PL84-99). Past performance information should be documented in a tabular format along with presentation on plans and profiles. In past performance tables, original descriptions should be included, along with any comments or assessments on the original information. An example of existing levee performance history documentation is included in Appendix C.

2.4.10 Existing Levee Improvement Measures. Information on past levee improvement measures is important to understand levee performance and remedial actions that were performed. Existing levee improvement measures may include but are not limited to levee raises, levee widenings, cutoff walls, seepage and stability berms, blankets, and relief wells. Design and as-built documents for these features should be collected. In many cases, past mitigation measures could be temporary measures after a flood event, and anticipated effectiveness of such measures should be evaluated for future events. Pre- and post-construction performance history should be considered to evaluate effectiveness of these measures.

2.5 Field Reconnaissance and Survey.

2.5.1 The field reconnaissance and survey should begin after the investigation team has become familiar with the area through the office study. The team should perform a field reconnaissance and survey of the proposed alignment or the existing levee and visit any proposed borrow areas. Observations made during site reconnaissance should be documented in detailed notes, supplemented by photographs and global positioning system (GPS) coordinates. The information, collected and considered during the office study, should be compared to observations from the field reconnaissance and survey to identify any major changes that may require further data collection such as an erosional feature. Local people or organizations in the area with knowledge of the levee (e.g., its construction, past performance, and geology) should be interviewed. During the evaluation and feasibility study phases, site reconnaissance and discussions with levee maintenance personnel and historians should be performed.

2.5.2 The following is a partial list of items that should be included in a field reconnaissance and survey:

- Levee alignment and centerline top of levee elevation
- Levee geometry including any localized steeper and narrower sections
- Topographic features
- Waterside conditions observed in natural berms and channel banks including exposed subsurface soil layers
- Signs of erosion or channel migration
- Surficial materials
- Drainage conditions (particularly poorly drained areas)
- Slope instability such as slumping, bulging at toe, or cracks
- Natural physiographic or landform features
- Man-made features such as pits, slope cut, ditches, posts, buried utilities, old fills, drain tiles, septic tanks, retaining walls, basements, and swimming pools
- Factors that may affect seepage conditions (e.g., entrance and exit conditions and blanket thickness, soil type, etc.).
- Vegetation and animal burrow conditions that may affect the levee performance
- Site-specific conditions that may affect levee performance during high water events

2.5.3 Topography and Bathymetry. Topography of a levee system may be useful in evaluating levee cross-sections (crown width, landside slope, and waterside slope), landside features (e.g., localized depressions), waterside features (natural berms, erosional features, river or stream bends), and land use (e.g., roadways and ditches). Bathymetry could be useful in evaluation of erosional features and seepage entrance conditions. A separate set of topography and bathymetry maps may be required based on project-specific goals. However, important information from topography and bathymetry should be part of plans and profiles.

2.5.4 Observations made during the field survey should be considered in developing reach boundaries and analysis cross-sections, as described in Chapter 5. These observations should be compared with previously existing field surveys, subsurface investigations, topographic, bathymetric, and geologic information to perform preliminary validation, or correction if needed, of the existing information and also to identify need for further data collection. For example, if the field study indicates erosional and stability features that are not captured in the current field surveys, a supplemental field survey may be required for the project.

2.6 Data Gap Evaluation. After an evaluation of existing data and field survey findings, the investigation and interpretation team should perform a data gap evaluation to evaluate the adequacy and quality of data that is available and to identify the types and extent of data that should be collected for use in different phases of a project.

Section II
Subsurface Investigations

2.7 General.

2.7.1 As stated previously, the subsurface investigations for a levee project are accomplished in different project phases. However, the standard of care for performing these investigations should be consistent in all phases of a project. For example, if an subsurface investigation such as a boring with standard penetration tests is performed during the feasibility phase of a project, the subsurface investigation should be performed in such a way that collected data could be used in all phases of the project including design phase without requiring significant additional efforts at the same location.

2.7.2 Subsurface investigations during the evaluation and feasibility studies should be limited and designed to obtain data needed to perform these studies. For example, if an absence of subsurface data in a reach influences decisions regarding future design work, subsurface data should be collected or the need for future investigation should be identified. Generally, the most comprehensive subsurface investigation efforts should be performed during the design phases.

2.7.3 Subsurface investigations during and after construction should be mitigation-measure-specific and should be performed to confirm design assumptions during construction and to monitor performance such as exploration to install piezometers. Observations, quality assurance testing, and quality control testing results should be documented properly for future assessments.

2.7.4 The purposes of subsurface investigations should include but not be limited to the following:

- Developing reach and sub-reach boundaries
- Characterizing the embankment and foundation for assessment of different potential failure modes and levee performance
- Obtaining data about anticipated weak areas of a reach or sub-reach, as identified from data review (including geologic, topographic, and bathymetric; physical features; and past performance problem areas). These areas may act as weak links in a chain link analogy.
- Obtaining geological and geotechnical data to support mitigation measures or new levee design
- Obtaining data about the areas near important utilities that may impact performance of levees during high water events

2.7.5 When explorations penetrate into or through an embankment, requirements in ER 1110-1-1807 shall be followed. This regulation establishes policy and requirements and provides guidance for drilling in dam and levee earth embankments and/or their earth and rock foundations. The primary purpose of this regulation is to prevent damage to embankments and their foundations from hydraulic fracturing, erosion, filter/drain contamination, heave, or other mechanisms during drilling operations, sampling, in-situ testing, grouting, instrumentation installation, borehole completion, and borehole abandonment. In particular, the exploration team should evaluate whether exploration techniques, such as rotary wash drilling, can cause damage to the levee and/or its foundation. Precautions should be taken, such as casing upper portions of the holes or selective degrading followed by regrading of the levee crest to prevent hydraulic fracturing.

2.8 Rationale for Subsurface Investigations.

2.8.1 A subsurface investigation program should be developed considering potential failure modes, site-specific conditions, cost, and coordination efforts required. A written comprehensive plan should be developed to justify the selection of exploration techniques, locations, sampling plan, and depths. Table 2-1 presents a partial list of factors that should be considered in developing a subsurface investigation program.

2.8.2 The exploration locations selected should be documented in a tabular format with rationale for each location exploration type, depth, and sample collection intervals. An exploration location map should be developed to show the locations of the proposed explorations. An example of an exploration rationale table is presented in Table 2-2.

Table 2-1. Some Considerations for Subsurface Investigation Program.

Factor	Comment
Limited information	There is little or no existing data or information with respect to levee design, construction, and performance.
Construction history and embankment conditions	Construction history indicates poor construction methods (i.e., dredged materials placed with limited or no compaction effort) and material types (i.e., materials prone to erosion, dispersion, through-seepage, and slope instability).
Past performance	Levee performance problems in past high water events, such as excessive seepage/boils, instability, or erosion.
Levee height	A taller levee (e.g., taller than 12 to 15 feet) that may be subject to higher hydraulic loading and may have greater consequences due to potential for deeper inundation.
Levee slopes	A steeper waterside slope may indicate existing erosion or potential for rapid-drawdown instability and erosion. A steeper landside slope may indicate potential for slope instability in high water events. Also, steeper slopes and narrow crown width may indicate a shorter seepage path in the levee and foundation.
Foundation conditions (geology and geomorphology)	Foundation conditions that may affect the levee performance such as: • Foundation soils are weak and compressible. • Foundation soils are highly variable along the alignment. • Foundation soils conducive to underseepage issues (i.e., locations of recent channel deposits, or continuity of aquiclude (e.g., impervious) layer). • Foundation soils that consist of unknown or highly variable man-made fill materials. • Foundation soil susceptible to liquefaction.
Topography and bathymetry	Topographic depressions on landside may indicate reduced blanket thickness. Also, bathymetry may indicate an adverse seepage entrance condition or erosional features.

Factor	Comment
Duration of high-water events	A high-water level for relatively long duration may indicate a higher potential for poor levee performance. Conditions should be evaluated for potential durations that have a higher potential for adverse effects on the levee performance. For example, a poorly maintained levee (i.e., no vegetation cover and/or subject to extensive animal burrowing) may experience poor levee performance at a shorter duration.
Borrow materials	Borrow materials for a proposed levee project should be evaluated to check suitability of materials, need for any special treatment before placement or design modifications, and specifications requirements.
Structures and utilities	Subsurface characterizations should include considerations for structures (e.g., bridges, railroads, tunnels, etc.) and major utilities that may have an anomalous condition compared to rest of the reaches.
Extent of mitigation measures	The lateral extent of the proposed levee project should be considered. Explorations may also be needed at transition areas between adjacent reaches selected for mitigation to evaluate potential three-dimensional effects between reaches.

Table 2-2. Example Rationale for Subsurface Exploration Program.

Exploration ID	Location and GIS Coordinates	Rationale for Exploration Location	Exploration Type, Depth, and Sample Frequency	Rationale for Exploration Type, Depth, and Sample Frequency
Boring 1	Location info such as crown, landside toe/field, waterside toe/field	Description of one or multiple factors for selecting exploration location	- SPT/CPT/Vane Shear, etc. - Sample frequency for SPT - Anticipated locations of undisturbed soil samples - Plan for data collection from CPT (blanket, aquifer, aquiclude, etc.) - Plan for pore water pressure dissipation test for CPT (one or multiple depths, anticipated locations) - Depths based on levee height and thickness of potential blanket, aquifer, and aquiclude layers and engineering properties such as thickness of soft soils - Insitu water level evaluations	- Rationale for selecting exploration type considering objectives such as borings as primary exploration and at critical locations, CPT as a paired exploration with borings or at gaps between borings, vane shear for shear strength estimation

2.9 Exploration Type, Spacing, Depth, and Sampling Frequency.

2.9.1 Exploration Type. Several different exploration techniques can be used for levee subsurface investigations include, but are not limited to, borings (SPT and/or undisturbed sampling), CPTs, vane shear testing, test pits, and geophysical data. Borings are advantageous in that they can be conducted rapidly and samples are obtained for visual identification, laboratory soil classification, and water content determination. However, for evaluating the engineering properties of fine-grained soils, such as consolidation and strength characteristics of soils, undisturbed sampling techniques are preferable. Auger borings and test pits (without SPT sampling) can be used to obtain bag and jar samples for testing. Hand augers allow for shallow sampling in areas inaccessible to a drill rig and also can provide samples for testing.

2.9.1.1 Trenches. Trenches are occasionally useful in borrow areas and levee foundations. CPTs are useful when paired with borings for extending the boring data to intermediate locations based on site-specific correlations between CPT signatures and adjacent soil borings. Sonic drilling and Becker hammer testing may also be necessary in areas with dense and large sized particles. Additionally, sonic drilling is a viable exploration type to reduce fluid usage through an embankment to comply with ER 1110-1-1807. All exploration penetrations into or through either the levee embankment or foundation layer should be properly abandoned such that any preferred path of seepage created by the penetration is properly mitigated.

2.9.1.2 Borings. Borings are typically classified as disturbed and undisturbed when conducted at a levee project. Borings are frequently used for more than one purpose, and it is not uncommon to use a boring for purposes not contemplated when it was made. Thus, it is important to have a complete log of each boring, even if there may not be an immediate use for some of the information. More information on what is required for a boring log is included in Section 2.13.10. There are many different methods used to perform a boring including auger boring (which act as a temporary casing though not watertight), drive boring (typically used to collect SPT samples and include Becker hammer testing), cone penetration boring (used to collect CPTs), and undisturbed boring (Shelby tube samples). More information on these boring methods can be found in EM 1110-1-1804.

2.9.1.2.1 The reliability of boring logs should be considered in interpretation and analysis. Field estimates and laboratory test results should be compared to evaluate consistency in field observation and logging. If the consistency evaluation indicates a poor correlation between field observations and laboratory testing results, field logs should be reevaluated using the collected samples. In such cases, additional index testing at critical layers may also be required to confirm soil classifications. Also, considerations should include whether the material exhibits stratification in the field that is obliterated by sampling and laboratory testing procedures (for example, fine interbedded layers of silt and sand mixed in sample bags when removed from split spoon samplers). In such cases, field identifications may be more appropriate for designating intermingled differing Unified Soil Classification System (USCS) material types. These interlayer materials are likely to exhibit anisotropic properties that may impact design. The following data should be included in profiles.

2.9.1.2.2 Additional laboratory test results such as dry, moist, saturated unit weights, over-consolidation ratio (OCR), cohesion intercept and drained friction angle, undrained shear strength, hydraulic conductivity, coefficient of uniformity, organic content should be reported in the boring logs and analysis cross-sections, as appropriate. Other observations or characteristics that may aid interpretation (cremation, loss of drilling fluid, drill rig chatter, change in penetration resistance between boring locations) should be documented in boring logs. Addition of these parameters may not be possible in profiles due to space limitations. Engineering judgement should be used to decide whether any of these parameters should be included on the interpretation figures.

2.9.1.3 Test pits. Test pits are a viable means for borrow site investigations as geotechnical characterization of a larger area within the borrow site can be performed in a cost effective manner. Test pits can be along the centerline of the levee/centerline of the inspection or observation trench to better explore levee foundation in areas of high uncertainty of soil conditions, subsurface debris, or critical layer contacts.

2.9.1.4 Cone Penetration Tests (CPTs). CPTs provide continuous cone and frictional resistance and can provide pore pressure measurements and information about soil stratification, strength, water table levels, and hydraulic conductivities. CPTs should be used as supplementary explorations when site-specific correlations with material types and laboratory test results from samples from borings have been established. Some geologic conditions may not be suitable for CPTs, such as layers of dense gravels or cobbles. Also, CPT data in dry soil above the water table or cemented soil may be difficult to correlate with boring findings, and caution should be used in interpretation for engineering evaluation.

2.9.1.4.1 Soil behavior types (SBT or SBTn) determined with state of practice procedures can be used to assist in evaluation of CPT data: however, USCS soil classification based on data collected during CPTs and samples from nearby borings should be used as the primary source of soil layer material type identifications. The raw data files from CPT testing should be retained in digital electronic form for future use.

2.9.1.5 Vane Shear Tests. Vane shear testing is a useful method for determining undrained shear strength, stress history, and sensitivity in soft clays, silts, and organic soils. Vane shear tests are not applicable in sandy soils or non-plastic silts as these may allow drainage during testing. Vane shear tests should be performed in accordance with ASTM D2573. It should be noted that vane shear tests should not be performed in sandy soils or non-plastic silts, as these may allow drainage during testing.

2.9.1.5.1 Field vane shear strength values require correction based on plasticity index (PI) values to determine mobilized shear strength as shown in Equation 5-1. Bjerrum's (1973a) correction factor, μ, has been commonly used to determine mobilized shear strength. ASTM D2573 also presents the vane shear correction factor, μ_R, as per Chandler (1988). The Chandler (1988) correction factor depends on time to failure along with PI. ASTM D2573 also lists Aas et al. (1986) as a reference for determining undrained shear strength. The SHANSEP procedure (Ladd and Foott 1974, Ladd et al. 1977, and Ladd 1991) can be used to determine maximum past pressure using undrained shear strength from the vane shear test. Sensitivity of soft soils can be determined as a ratio of peak and remolded shear strengths.

$$(S_u)_{fv}(mob) = \mu(S_u)_{fv} \tag{5-1}$$

where:
$(S_u)_{fv}(mob)$ = Mobilized undrained shear strength from field vane shear test
μ = Correction factor (Bjerrum 1973a, Chandler 1988, or Aas et al. 1986)
$(S_u)_{fv}$ = Undrained shear strength from field vane shear test

2.9.1.6 Geophysical Exploration. Geophysical methods may be useful in levee projects to evaluate subsurface conditions along with borings, CPTs, and other exploratory methods. Geophysical exploration may provide an estimate of continuity of geologic features (e.g., aquifers), and presence of anomalous conditions such as utilities and narrow geologic features. Geophysical methods may not be reliable in areas that have too many obstructions, such as utilities, bridges, or fences.

2.9.1.6.1 Geophysical methods should be selected based on site-specific conditions. Borings or CPTs may be required to confirm geologic features of interest such as aquifers, channel deposits, or thin low-resistivity layers potentially indicating presence of fine-grained soils. If geophysical studies are performed before exploratory investigations, the results of geophysical studies should be used to assist in selecting exploratory investigation locations. If geophysical studies are performed after an exploratory investigation program, supplemental confirmatory explorations may be needed.

2.9.1.6.2 Choosing appropriate geophysical methods can be challenging and is dependent on the physical characteristics of the site, the properties of the target being investigated, and the ability to measure a signal over the target of interest from the surrounding background values. Important considerations in the choice of these methods include the size of the area, depth, the underlying site geology and associated soils, budget available for study, and site access considerations. When evaluating whether to use a geophysical method, it is important to understand what type of information is being sought, the appropriateness of the method selected, the reliability of the results, and the cost versus benefit of the results. It is also important to understand the capabilities of the different geophysical methods so that they may be used appropriately for subsurface investigations. Table 2-1 in EM 1110-1-1802 provides a decision matrix of surficial geophysical methods for specific investigations. Table 4-1 in EM 1110-1-1804 provides applicability of geophysics to different engineering parameters. Geophysical data must be interpreted in conjunction with borings and by qualified, experienced personnel.

2.9.1.6.3 Commonly used geophysical methods have a range of applications and a range of ideal site conditions in which to conduct them. Seismic methods generally favor targeted subsurface layers with good contrast in material strengths and possibly saturation conditions. Electrical resistivity and electromagnetic methods are generally sensitive to subsurface layers with contrasting saturation conditions, contrasting soil gradations or mineralogy, or presence of air-filled cracking or large voids. Self-potential (SP) surveys are most successful when fresh seepage water is being mapped in a generally clean sand or gravel seepage path environment, or when seepage exits a clay embankment core into coarser-graded shell materials. Ground penetrating radar (GPR) surveys are generally most successful in a non- or low-electrically-conducting medium (i.e., clean sands, some silts, concrete, or ice). GPR surveys typically do not perform well in highly conductive clay environments because of high signal attenuation and resulting poor depth penetration of the transmitted signal.

2.9.1.6.4 The resolution of a given target varies widely among the different geophysical methods, and the solutions are often inverse solutions; that is, a causative physical condition (for example, seepage) is inferred from the field readings. Geophysical analysis results are usually non-unique; in order to choose from several valid models (interpretations) the interpreter typically must see how the various hypotheses conform to information from other supplemental geologic or engineering data. Geophysics rarely give reliable interpretations absent corroboration with other forms of subsurface data.

2.9.1.6.5 lists typical embankment deficiencies and exploration targets and the geophysical methods that have been successfully employed in evaluating those deficiencies and targets (CEATI 2005, Fell and Fry 2007, SEG 1990, SEG 2005, and EM 1110-1-1802). This table was originally prepared for applications in dams and therefore may include items that may not be as applicable to levees.

Table 2-3. Geophysical Survey Techniques for Typical Embankment Deficiencies and Exploration Targets.

Embankment deficiency or exploration target	Geophysical techniques to consider	Remarks
a. Anomalous or non-uniform seepage	1. Self-potential (SP) 2. Electrical Resistivity 3. Temperature	1. Seepage creates ‘streaming potential’ measured by SP survey. 2. Seepage may be more conductive or more resistive than adjacent soils. Usually used with SP. 3. Seepage may be colder or warmer than adjacent embankment materials. May require installation of fiber optic cables as sensors.
b. Possible areas of piping or internal erosion	1. Self-potential 2. Electrical Resistivity 3. Temperature	1, 2. Similar to an anomalous or non-uniform seepage. Note that a series of surveys over time may be needed to detect progression of piping/erosion. 3. Seepage may be colder or warmer than adjacent embankment materials, especially if piping has shortened effective seepage path length.
c. Foundation sands and gravels, lenses, bar deposits	1. Electrical Resistivity 2. Seismic methods 3. Self-potential (SP) 4. Ground penetrating radar (GPR)	1. Dry clean sands and gravels show high resistivity, while wet sands and gravels may show much lower resistivity. However, in comparison with fine-grained blanket layers, wet sand and gravel may still show higher resistivity than fine-grained blanket layers. 2. Seismic reflection may detect sands, gravels, if in buried channel deposits. 3. SP may indicate locations if deposits are conveying seepage. 4. GPR may delineate thin lenses if overlying material is not too conductive (clayey soils).
d. Piping or voids around concrete conduits, or beneath spillways	1. Ground penetrating radar (GPR) 2. Impact Echo (IE)	1. GPR imaging on the spillway or from inside the dewatered conduit is effective in detecting voids. Air-filled voids are easier to detect vs. water-filled voids. 2. Conducted from inside dewatered conduits. Impact echo reflections will generally be different over voids than over non-void areas.
e. Voids around metal (i.e., corrugated metal pipe) conduits	1. Temperature, Infra-red thermography 2. Impact Echo (IE); other acoustic measurements	1. Air-filled or water-filled voids may show anomalous temperature readings than sound backfill conditions. Thermograms (thermal images) are taken from inside the conduit. 2. IE frequency changes may indicate voids. Requires access to inside of conduit.

Embankment deficiency or exploration target	Geophysical techniques to consider	Remarks
f. Locations of ‘lost’ or concealed metallic pipes	1. Ground penetrating radar (GPR) 2. Magnetometer / Pipe locator	1. GPR penetration is site-dependent, may be limited in clay soils and high groundwater levels. Metallic pipes are readily detected by GPR beneath concrete slabs. 2. Iron or steel pipe within a few feet of the surface can often be located by ‘pipe locators’ or magnetometers. Energizing pipe at one end generally improves delineation by pipe locator.
g. Gradation changes along levees, changes in core/shell configurations	1. Resistivity / Electromagnetic profiling 2. Seismic surface wave; seismic refraction tomography	1. Resistivity changes may indicate change from clays to silts, sands, etc. 2. Seismic surface wave or refraction tomography may indicate core/shell changes or gradation changes.
h. Animal burrows and associated voids within levees or embankments	1. Electrical resistivity (ER) profiling 2. Ground penetrating radar (GPR) 3. Seismic refraction tomography profiling	1. ER profiling may indicate presence of air- or water-filled burrows. Electrode spacing may need to be close, as this is a site-dependent survey. 2. GPR imaging more effective in sands/silts; less effective in clays. Air-filled burrows easier to image than water-filled burrows. 3. Refraction tomography profiling along the crest may indicate presence of burrows/other voids. Site dependent similar to ER profiling described above (h).
i. Embankment fracturing, including desiccation cracking, differential settlement, subsidence	1. Seismic profiling 2. Resistivity profiling	1. Seismic shear-wave profiling generally is sensitive to locations of transverse cracking. High-resolution reflection may indicate offsets related to settlement or subsidence. 2. Resistivity profiling may indicate locations of air-filled transverse cracking.
j. Configuration of soil/bedrock contact (“top-of-rock” configuration)	1. Seismic profiling 2. Resistivity profiling 3. Ground penetrating radar (GPR)	1. Seismic refraction, seismic reflection, or surface wave may be used depending upon site conditions, geology, and groundwater (saturation) conditions. 2. Resistivity profiling may be used depending upon soil and rock types and saturation conditions. 3. GPR may delineate the ‘top-of-rock’ if the overlying soils are not too clayey.

Note: This list is not all-inclusive; it lists geophysical techniques commonly used for these targets or deficiencies.

2.9.2 Exploration Spacing.

2.9.2.1 The spacing of explorations should be based on results of the data collection and review (Sections 2.4 and 2.5), evaluation of the subsurface interpretation assessed in the previous exploration stages, from prior experience in the area, and/or evaluation of levee potential failure modes. The spacing of explorations should not be arbitrarily uniform but rather should be based on available geologic information and past performance. Spacing of explorations usually varies from 200 to 1,000 feet along the alignment, being more closely spaced in expected problem areas (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 (older geologic formations without past performance distress).

2.9.2.2 Combinations of explorations are normally laid out along the levee centerline, along the landside toe, and along the waterside toe alignments to evaluate changes in stratigraphy and stress conditions. An exploration located at a distance landward of the landside levee toe is recommended to be a part of the explorations set per location along the levee. It is often needed to verify blanket thickness and properties, ensure continuity of the blanket, and inform seepage control measure design. Engineering rationale and judgment for each exploration location should be summarized as described in Table 2-2. It is understood explorations may not always be feasible at preferred locations due to limited real estate, vegetation growth, or conflicts with existing environmental or cultural resources. Regardless of these constraints, designers should ensure they collect sufficient data for reliable design which may require additional right of way when necessary.

2.9.2.3 In general, ten to fifteen explorations (borings and/or CPTs) per mile (levee crest, landside, and/or waterside) are considered reasonable for levee design. However, the density and distribution of explorations should be project-specific and should incorporate considerations of geological characteristics, past performance, and failure modes. If CPTs are used, site-specific correlation with borings should be performed to verify the accuracy of the CPT results. Site-specific correlation typically requires one companion boring for every five CPTs. Once a site-specific correlation between boring sample material types and CPT data are established, CPT data can be used to evaluate geologic conditions between borings. However, borings should be selected for critical locations where visual confirmation of samples with index testing and other laboratory testing (strength and consolidation testing) are needed for evaluation and design.

2.9.3 Depth.

2.9.3.1 In general, the depth of explorations below the levee/foundation contact along the alignment should be no less than three times the levee height. Exploration depths should always be deep enough to provide data for engineering analyses (seepage and stability analyses) of the levees and to define the character of the subsurface (confining layers for aquifer, aquicludes, bedrock depth, etc.). This is especially important when the levee is located near the riverbank where explorations must provide data for stability analyses involving both levee foundation and riverbank. Where pervious or soft materials are encountered, explorations should extend through the permeable material (aquifer) to impervious material or through the soft material to firm material.

2.9.3.2 Explorations at structure locations should extend well below invert or foundation elevations and below the zone of significant influence created by the load. Explorations must be deep enough to permit analysis of approach and exit channel stability and of underseepage conditions at the structure.

2.9.3.3 The explorations should extend below the aquifer layer to assist in evaluating seepage and stability conditions. If multiple aquifers are separated by fine-grained layers, the continuity of the fine-grained layers should be investigated. Explorations for proposed cutoff walls should extend some minimal distance into the material in which the wall will terminate such as an aquiclude. If a thick aquifer requires very deep explorations (e.g., 150 feet), engineering judgment should be used to estimate the required exploration depths based on project-specific goals. In general, at least one exploration per mile should be advanced below the aquifer for such deep aquifer conditions. However, more frequent deeper explorations may be needed for cutoff wall design and construction. When high resolution geophysical data can be obtained at deeper depths, geophysical profiles may be useful in these cases. In borrow areas, the depth of exploration should extend several feet below the target borrow depth or to the groundwater table. If borrow is to be obtained from below the groundwater table, borings should be at least 10 feet below the bottom of the proposed excavation to support potential construction dewatering analyses.

2.9.4 Frequency of Sampling.

2.9.4.1 Final design will require information on soil layer thicknesses. Within each boring, soil sampling should be completed on a close enough frequency that contacts between different layers can be identified with reasonable accuracy. The contact between dissimilar soils can be located based on subtle and obvious changes in the drill rig action. The driller and the geologist should maintain close communication so that these drilling actions are communicated and entered onto the field log. In general, sample collection intervals should not exceed 5 feet. Samples should be collected after proper cleaning of cuttings off the bottom of the boreholes and as per ASTM D1586.

2.9.4.2 Vane shear testing is usually performed in target zones within suspected soft cohesive soil layers identified from prior explorations, levee construction history, and geologic information.

2.10 Undisturbed Soil Samples.

2.10.1 In addition to SPT sampling, undisturbed soil samples should be collected for laboratory testing for drained and undrained strengths and consolidation behavior. The undisturbed soil samples should generally be collected according to ASTM D1587. However, depending on local practice, a variety of piston samplers and core barrel samplers have also been used successfully to collect undisturbed soil samples (see EM 1110-1-1804).

2.10.2 A 3-inch diameter “Shelby” tube with a sharpened cutting edge is most commonly used for collecting undisturbed samples of fine-grained soils. In some situations, larger diameter tubes (e.g., 5-inch diameter) are used and will often result in reduced disturbance and provide a larger sample for testing. Undisturbed soil samples should be examined and evaluated for suitability for testing. The most undisturbed and representative portion of the sample should be used. When sample disturbance effects are significant and will result in a significant impact to the assessment of performance and risk, ASTM D4452 can be followed to perform X-ray radiography of soil samples within steel tubes, which can then be evaluated for disturbance or other features such as presence of fissures, inclusions, layering, or voids.

2.10.3 Preliminary assessment of strength parameters can aid with selecting locations of undisturbed soil samples, selecting locations of samples for laboratory testing, and assessment of stability conditions. Laboratory testing and slope stability analysis are discussed in Chapters 3 and 7 respectively. Table 2-4 provides a list of preliminary field strength estimating methods which can supplement information developed from laboratory testing and detailed engineering evaluations.

Table 2-4. Preliminary Appraisal of Foundation Strengths from In-Situ Tests.

<table>
<tr><th>Method</th><th>Remarks</th></tr>
<tr><td>1. Standard Penetration Testing (SPT)</td><td>a. Undrained shear strength of fine-grained soils can be estimated using SHANSEP (Stress History and Normalized Soil Engineering Properties) method (Ladd and Foott 1974, Ladd et al. 1977, and Ladd 1991) using the following relationship:

$Su = \sigma_v' \ S \ (OCR)^m$, where $S = (Su/\sigma_v')_{nc}$, which is usually between 0.22 and 0.25; m is an exponent between 0.75 and 0.85, and a preliminary assessment of OCR value would be needed for preliminary appraisal of undrained strength.

SPT N-values are not reliable to obtain undrained shear strength of soils. However, in absence of laboratory testing, the following relationship (Terzaghi and Peck 1967) could be used to perform preliminary engineering assessment.

<table><tr><th>SPT N_{60}</th><th>Soil Consistency</th></tr><tr><td>< 2</td><td>Very Soft</td></tr><tr><td>2 – 4</td><td>Soft</td></tr><tr><td>4 – 8</td><td>Medium</td></tr><tr><td>8 – 15</td><td>Stiff</td></tr><tr><td>15 – 30</td><td>Very Stiff</td></tr><tr><td>> 30</td><td>Hard</td></tr></table>
b. Drained friction angle of coarse-grained soils can be estimated using the Hatanaka and Uchida (1996) relationship:
$\phi' = [15.4 \ (N_1)_{60}]^{0.5} + 20$
Other state-of-practice relationships for estimating drained friction angle could be utilized also.</td></tr>
<tr><td>2. Natural water content</td><td>Useful when considered with soil classification and previous experience. Note that when using water content to assess strength, some soils will erroneously appear to have a high strength if dry, which would change with increased saturation level. For example, a clay sample (CH) may appear to have high strength when very dry but not at higher water contents.</td></tr>
<tr><td>3. Position of natural water contents relative to liquid and plastic limits</td><td>Useful where previous experience is available. If natural water content is close to plastic limit, foundation shear strength should be high at its current state. Natural water contents near liquid limit indicate sensitive soil usually with lower shear strengths.</td></tr>
<tr><td>4. Torvane or pocket penetrometer tests on intact portions of general samples or on walls of test trenches</td><td>Easily performed and inexpensive but may underestimate actual values; useful only for preliminary strength classifications. Intact portions of samples may not represent properties for the soil continuum.</td></tr>
<tr><td>5. Cone penetration test tip resistance</td><td>Undrained shear strength (S_u) is related to cone tip resistance. Undrained shear strength from cone penetration test results should be calibrated to local correlations using laboratory test and field data.</td></tr>
<tr><td>6. Vane shear test</td><td>Peak and remolded undrained shear strength after applying correction factors.</td></tr>
</table>

2.11 <u>Hydraulic Conductivity Estimates</u>.

2.11.1 The hydraulic conductivity of pervious foundation materials can often be estimated with reasonable accuracy by using empirical correlations of hydraulic conductivity with grain size distribution and relative density of soils. EM 1110-1-1804 has a comprehensive list of the methods to conduct hydraulic conductivity estimates. Field pumping tests are often the most accurate means of determining hydraulic conductivity of stratified in-situ pervious deposits or aquifer deposits. Field pumping tests are expensive and usually justified only at sites of important structures and where a large number of pressure-relief wells are planned. Due to their cost, they are recommended only when the geology and stratigraphy of the subsurface conditions are known The general procedure is to install a well and install piezometers at various distances from the well to monitor the resulting drawdown during pumping of the well. Appendix C of Army Technical Manual TM 5-818-5 (1983) and EM 1110-2-1914 include procedures for performing field pumping tests.

2.11.2 The differences between hydraulic conductivity measured in the field and in the laboratory remain a challenge for the design of levee seepage measures. In the Mississippi Valley, there is generally a long history of field testing correlated with in-situ confirmation of pore pressures and related flow volumes, such as from relief wells (DIVR 1110-1-400 Tables 1 and 4). In other watersheds across the Nation, where there is less field-confirmed performance data, hydraulic conductivity may be more accurately estimated using laboratory tests. In general, the hydraulic conductivities of fine-grained soils from the laboratory tests are typically lower than field-estimated values, often by one or two orders of magnitude.

2.11.3 The difference between laboratory and field hydraulic conductivity values is more evident in semi-pervious to impervious blankets. The overall hydraulic conductivity of blanket layers is affected by natural and man-made defects such as presence of fissures, root holes, former sand boil holes, and other perforations. According to USACE studies by Mansur and Kaufman (TM 3-424 1956), the back-calculations of field behavior showed that clay blankets had effective hydraulic conductivity much higher than predicted by laboratory tests (up to 1000 times) and were commonly just as pervious as silt blankets. Also, a significantly lower hydraulic conductivity of a blanket layer without considerations of these natural and man-made defects may artificially show a higher exit gradient in seepage analyses, as exit gradient depends on the ratio of the horizontal hydraulic conductivity of the aquifer to vertical hydraulic conductivity of the blanket layer ($k_{h(aquifer)}/k_{v(blanket)}$), among other factors.

2.11.4 The California Department of Water Resources (DWR) documented hydraulic conductivity values of different types of soil from laboratory and field testing from investigation of levees in California (URS 2015). Figure 2-1 shows variations of vertical hydraulic conductivity values with fines content from the DWR study and can be used to estimate ranges of hydraulic conductivity values from laboratory and field testing. These hydraulic conductivity ranges match well with previous studies and publications and are considered fairly reliable for mineral soils deposited in geomorphic environments similar to those in the California Central Valley. The ranges may not be very reliable for estimating values for peats, organic materials, and horizontal permeability.

2.11.5 Empirical equations such as Kozeny-Carman and Hazen equations have been modified recently to account for both relative density and grain size distributions of soils. These equations should be used to develop site-specific hydraulic conductivity values for levee projects. The site-specific values should be compared with values shown in Figure 2-1 to evaluate the reasonableness of selected values. A modified version of Kozeny-Carman equation, as proposed by Chapuis and Aubertin (2003a) requires a complete gradation curve (both sieve and hydrometer), as it accounts for the surface area of the particles. In absence of hydrometer tests, the estimated hydraulic conductivity value (k) could be higher, as the tail portion of the grain size distribution impacts the estimate. Chapuis (2004) provides a relationship for hydraulic conductivity values based on void ratio, e and d_{10} (mm) values, which is also considered a modification of the original Hazen (1911) equation. This predictive equation (Chapuis 2004) was compared with published results for sand and gravel specimens, with an effective diameter (d_{10}) between 0.13 and 1.98 mm and a void ratio (e) between 0.4 and 1.5. The Chapuis (2004) equation is shown in Equation 2-1.

$$k(cm/s) = 2.4622[d_{10}^2 e^3/(1+e)]^{0.7825} \tag{2-1}$$

where:

k = hydraulic conductivity (cm/s)
d_{10}= effective diameter (mm)
e = void ratio

2.11.6 Absolute values of hydraulic conductivity are difficult to determine, and these may not be applicable across a soil layer. However, relative hydraulic conductivity values of blankets (also referred to as top stratum) and aquifers (also referred to as substratum) should be estimated such that it provides a representative computational contrast between a blanket layer with lower vertical hydraulic conductivity and an aquifer layer with higher horizontal hydraulic conductivity [$k_{h(aquifer)}/k_{v(blanket)}$]. As long as a reasonable relative hydraulic conductivity values are estimated based on empirical equations and within generally accepted range of values, the seepage analyses using relative hydraulic conductivity values will provide reasonable estimates of pore water pressure distributions.

2.11.7 Underestimation of absolute hydraulic conductivity of an aquifer layer will result in an underestimate of seepage flow quantities, resulting in underestimates of required discharges for pressure relief measures, such as relief wells, toe drains, and relief trenches. When designing a pressure relief component, the designer should make special efforts to confirm that the estimates of hydraulic conductivity are consistent with expected field performance and that relief structures are sufficiently large to handle required discharge rates to achieve required pore pressure relief.

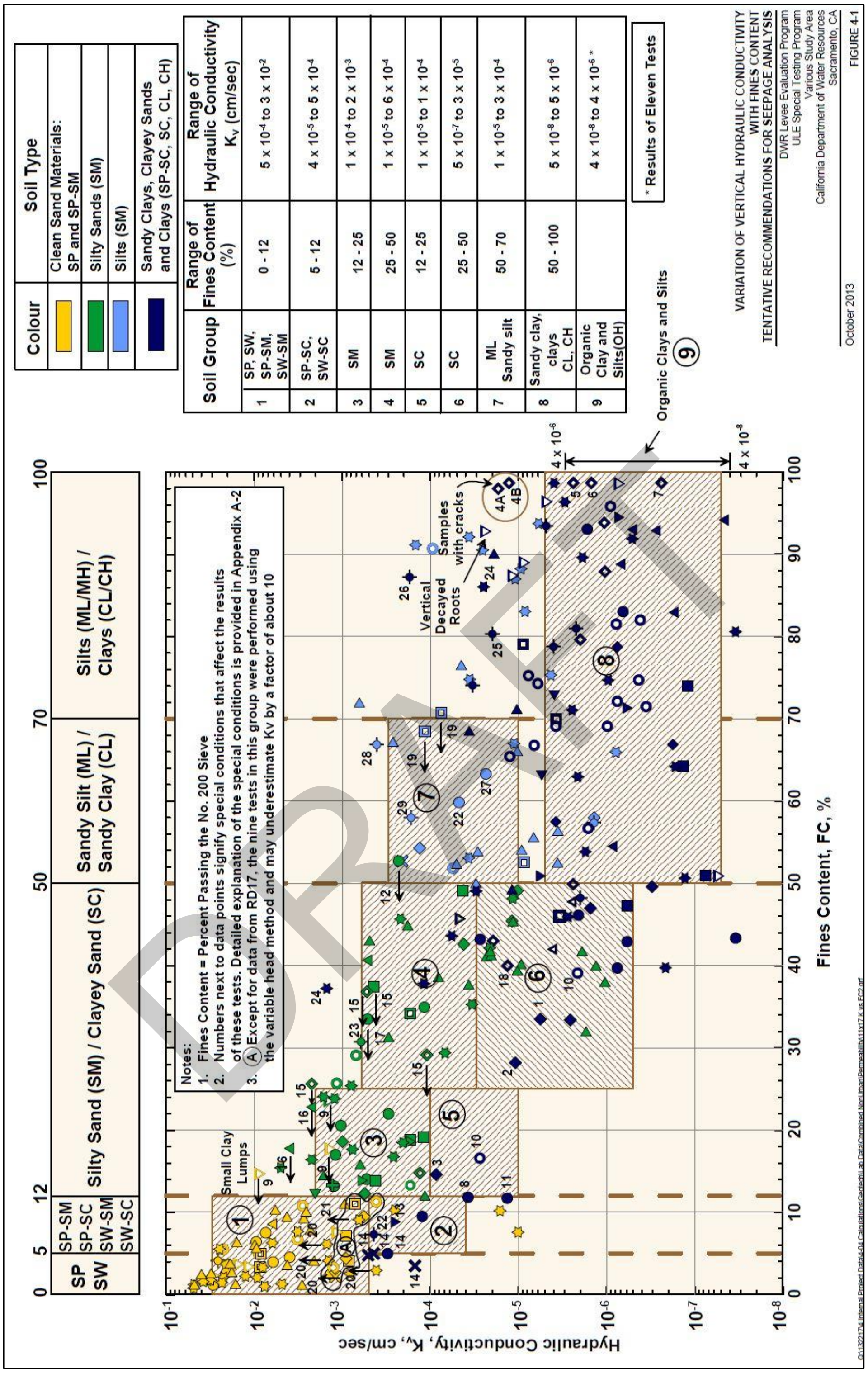


Figure 2-1. Variations of Vertical Hydraulic Conductivity Values with Fines Content (Tentative Ranges) Developed by CA DWR (URS 2015).

2.12 Groundwater and Pore Pressure Observations.

2.12.1 Groundwater levels, if encountered, should be measured during explorations (borings, CPTs, or other exploratory methods). Groundwater levels measured during exploration are important for constructability evaluations and engineering analysis. It should be recognized that groundwater levels vary over time and measurements obtained during explorations may not be reflective of these changes without long term monitoring.

2.12.2 Groundwater elevations, as measured from piezometers, monitoring wells, relief wells, dissipation tests during CPT, observations from borings, falling head tests, and other sources should be collected. Groundwater levels should be reported along with date of measurement and water surface elevations in the adjacent lake, river, channel, canal, or ditch. If gage station readings are included, distance from the groundwater monitoring locations should be included.

2.12.3 Piezometers can provide useful information on levee performance and should be utilized to evaluate the underseepage conditions of a levee. Piezometers can be installed during the feasibility or design phases to provide long-term monitoring of groundwater conditions and levee performance. Piezometers can also be used to monitor long term performance of mitigation measures. The use and installation of piezometers are described in EM 1110-2-1908. Hydraulic conductivity tests should be performed after installation of the piezometers; these tests provide information on foundation hydraulic conductivity and show if piezometers are functioning. Testing and interpretation procedures are described in EM 1110-2-1908.

2.13 Subsurface Explorations for Low Consequence Levees. Levees that have low life-safety and economic consequences in an event a levee breach occurs may not warrant extensive subsurface investigations and engineering analyses to develop and implement mitigation measures. In some situations, regional engineering guidelines, if available, can instead be considered. For example, the California Department of Water Resources has published a set of engineering guidelines and templates for levees in rural areas of the Central Valley of California (California Department of Water Resources 2014).

CHAPTER 3

Laboratory Testing for Levees

3.1 General. This chapter discusses the laboratory testing of soils related to the levee embankment, levee earthen design features (seepage and stability berms), and the underlying foundation. The laboratory testing of structural features related to levees (such as floodwalls, pipes, or concrete walls) should adhere to USACE standards covered in other engineering manuals. Laboratory testing programs for levees will vary from minimal to extensive, depending on the nature of the project, how well the foundation conditions and borrow materials are known and whether existing experience and correlations are applicable. Any testing program should be carefully crafted to reliably answer particular questions and consider both the short and long-term use of the data. The laboratory testing program should be developed in concert with applicable field testing by such methods as SPT samples, CPT soundings, and Shelby tube soil samples. At a minimum, testing programs generally consist of water content, unit weight, and soil classification tests on most soil samples. More expensive and time consuming tests may be needed to identify pertinent soil properties such as shear strength, consolidation characteristics, hydraulic conductivity, and compaction characteristics. The tests should be conducted on representative samples and generally under laboratory conditions that are representative of operating conditions for the design. The designer is encouraged to make use of all available data from geological studies when selecting representative samples for testing. Laboratory tests should follow the most current American Society of Testing and Materials (ASTM) standards. A summary of common laboratory tests conducted for levee design and construction activities is provided in Table 3-1. It should be understood that ASTM standards will change over time and that the standards referenced in this report may be modified or replaced or become obsolete. Designers should always use the most current and applicable ASTM standard. Special tests (not included in ASTM standards) may also be required to assess specific material properties that are less commonly required, such as soil erodibility, moisture suction, and unsaturated hydraulic conductivity relationships.

3.1.1 Laboratory and Field Testing Facilities and Operations. Every facility performing laboratory testing must be validated as an approved testing laboratory as per ER 1110-1-8100, Laboratory Investigations and Testing, and ER 1110-1-261, Quality Assurance of Laboratory Testing Procedures. The validation procedures should be in accordance with ASTM E329, Agencies Engaged in Construction Inspection, Testing, or Special Inspection, and ASTM D3740, Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction. The validation is specific to certain laboratory and field tests and the facility should be validated for the specific laboratory and field tests it is requested to perform. Temporary laboratory testing facilities may also be established near the work area for large laboratory and field testing programs. These temporary laboratory testing facilities should be established on a ground floor or basement with a solid floor and shall be free of traffic and machinery vibrations. Separate areas may be designated for dust-producing activities such as sieve analyses and sample processing. Temperature control of the entire laboratory is preferred. If the temperature-controlled space is limited, this space should be used for triaxial compression, hydrometer, specific gravity, consolidation, and hydraulic conductivity testing. A humid room, large enough to permit the storage of samples and the preparation of test specimens, is preferable. Testing equipment used in the laboratory should be calibrated to confirm it meets the appropriate calibration standards.

Table 3-1. Common Laboratory Tests for Levees (note – not all tests may apply) [Note 1]

Test	ASTM Standard
Soil Classification and Index tests	
Unified Soil Classification (USCS)	D2487-17e1 (testing/laboratory identification) and D2488-09a (visual/field identification)
Water Content	D2216-19 (recognized standard)
Grain Size	D6913/D6913M-17 (sieve analysis) D7928-21e1 (Hydrometer) D1140-17 (amount finer than No. 200 sieve by washing)
Atterberg Limits	D4318-17e1
Specific Gravity	D854-14
Organic Content	D2974-20e1 (Loss on ignition test)
Unit Weight and Void Ratio	D7263-21
Dispersion Test	D4221-18 (Double hydrometer test) D6572 (Crumb test) D4647 (Pinhole erosion test)
Soil Hydraulic Conductivity tests	
Coarse-grained Soils	D2434-19
Fine-grained Soils	D5084-16a
Soil Consolidation test [Note 2]	
Incremental Load Method	D2435 / D2435M-11(2020)
Constant Rate-of-Strain	D4186 / D4186M-20e1
Soil Compaction test	
Standard Proctor	D698-12(2021)
Modified Proctor	D1557-12
Soil Shear Strength tests	
Unconfined Compression	D2166 / D2166M-16
Triaxial Compression	D2850-15 (Unconsolidated, Undrained) D4767-11(2020) (Consolidated, Undrained) D7181-20 (Consolidated, Drained)
Direct Shear	D3080/D3080M-11 [Note 3]
Laboratory Miniature Vane Shear	D4648/D4648M-16
Direct Simple Shear	D6528-17
Fully Softened Shear Strength	D7608-18e1

Note 1: ASTM standards are routinely modified and are sometimes removed and replaced. Designers should always use the most applicable and current ASTM standard.
Note 2: These tests can also be used to evaluate permeability of fine-grained soils.
Note 3: The Direct Shear test was withdrawn by ASTM in 2020. There is currently no replacement test and it should continue to be used until a replacement test is approved by ASTM.

3.1.2 Quality Assurance and Quality Control. Quality Assurance (QA) and Quality Control (QC) procedures should be developed and implemented for laboratory testing programs. These procedures should be comparable to industry standards (ASTM) and follow ER 1110-1-8100, ER 1180-1-6, and ER 1110-1-261.

3.1.3 Reporting of Laboratory Test Results.

3.1.3.1 At a minimum, laboratory test results should be reported following requirements found within ASTM D3740 and within each respective ASTM test method. The results of laboratory tests should be reviewed by the designer and engineer as soon as practical after the completion of the test to verify the results and make adjustments to the laboratory testing program as necessary. The reasonableness of the test results should be evaluated considering the field observations/testing and in situ testing. Laboratory and field testing results should be complementary and any differences should be explainable. Additional laboratory testing may be required if discrepancies between laboratory test results, field observations, or in-situ testing cannot be explained. Generally, the following data should be included on the laboratory testing reports:

- Name and address of the testing laboratory.
- Identification of the report and the date issued
- Identification of the project
- Description of the test sample
    - Boring name/number or trench name/number
    - Type of sample: disturbed, undisturbed, general, compacted, bulk, etc.
    - Discussion of any methods used to evaluate sample integrity (e.g. X-ray of tube)
    - Method of sample extraction from sampling medium (e.g. Shelby tube) and condition of tube
    - Description of quality, features or defects of the test specimen
    - Photographs of laboratory specimen used for undisturbed testing (e.g. shear strength) before and after testing
- Identification of the test sample.
    - Sample number or ID
    - Date sample was collected
    - Coordinates (latitude, longitude, elevation) of sample
    - Depth interval for sample (include elevation interval if available and reliable)
    - Project stationing and offset from centerline
    - Lift number (if applicable)
    - Date of receipt of the test sample
- Date(s) test(s) performed
- Identification of the standard test method(s) used and a notation of deviations from the standard
- Test results and other pertinent data required by the standard test method
- Any additional sample and field identification/location information.

3.1.3.2 Laboratory test data should be stored in a geotechnical database library in order to maintain the data and produce boring logs under graphical standards as described in Chapter 5.

3.1.4 Use of Correlations. The use of correlations in development of engineering properties of soils for levee design and evaluation can be used to augment laboratory testing and in-situ testing programs. Correlations using less costly index testing combined with in-situ testing to assess shear strengths, consolidation properties, and permeabilities for levees should be used where appropriate. The proper use of correlations requires a great deal of judgment and experience. Correlations are often limited to specific geological formations or to a specific area. Thus, soils within the work area may not be represented by the soils used in the correlations due to differences in soil mineralogy, fabric structure, soil grain shape, geological history, and soil grain texture. If properties assessed from correlations are critical, then the rationale for use of these correlations for the in-situ should be documented.

3.2 Soil Classification.

3.2.1 Visual classification of all soil samples from the levee embankment, levee foundation, and levee design features should be conducted (ASTM D2488) during field investigations and in the laboratory during testing. Conflicts between the field visual classifications, laboratory visual classifications, and laboratory testing results should be resolved and documented during development of the final boring logs. Generally, Unified Soil Classification System (USCS) per ASTM D2487 will then be used to present the soil classifications shown on the final boring logs. Local variations of the USCS are used in some areas, and may be necessary and appropriate (for example, more refinement of plasticity related to variations in strength and consolidation characteristics may be warranted). If local variations are used, the procedure should be properly documented and justified, and then consistently used. All data recorded during the classification process should be recorded on appropriate forms. Recorded data should include but not be limited to strata elevations, soil visual description, soil type, moisture content, consistency, color (preferably using a standard scheme such as the Munsell system), and modifiers. Water content determinations are typically performed on all samples except clean sands and gravels.

3.2.2 Visual classifications should be verified with laboratory testing on a sufficient number of samples to properly characterize site conditions. Laboratory verification of soil classification requires a determination of grain size or particle size and Atterberg Limits. Per ASTM D2487, proper soil classification should generally include appropriate grain-size test(s) (ASTM D6913, ASTM D7928, and ASTM D1140) for both fine-grained and coarse-grained material. The proper USCS classification for fine-grained material will require Atterberg Limits (ASTM D4318) when appropriate.

3.2.3 While organic content is not required for soil classification, testing of organic content is often used to support soil classification, consolidation, and strength testing of materials with high organic contents. Organic content per ASTM D2974 should be quantified with Loss on Ignition tests on specimens. Soils classified as peat in accordance with ASTM D2487, can further be classified using ASTM D4427.

3.3 Soil Consolidation.

3.3.1 Consolidation tests are used to assess the soil stress history (including preconsolidation pressure and over-consolidation ratio) and/or to establish parameters to estimate settlement. Generally, the following conditions increase the need for good quality data to make accurate settlement predictions:

- Foundation clays are highly compressible
- Foundation soils under high levees are somewhat compressible
- Structures (e.g., drainage pipes, gate wells, floodwall transitions, etc.) within the levee system are sensitive to settlement and differential settlement

3.3.2 Refer to Chapter 8 for more details on settlement analysis and settlement design procedures.

3.3.3 Consolidation testing should be performed using either the Incremental Load Method (ASTM D2435) or the constant rate-of-strain method (ASTM D4186). The incremental load method is the more commonly used method which does not require the same equipment (i.e., for back pressure saturation) as the constant rate-of-strain method. Specific gravity tests, conducted in accordance with ASTM D854, should be completed on all consolidation test specimens that are obtained from a geologic setting where designers have limited engineering experience or from areas where variations in specific gravity can be significant. Assumed specific gravity values may be used when they can be applied with a reasonable degree of confidence. Test reports from consolidation tests should include but not be limited to the boring name, sample visual description, sample elevation, sample location, grain size, Atterberg Limits, specific gravity (measured or assumed), water content, dry density, saturation, initial void ratio, compression index (C_c), recompression index (C_r), diameter and height of the sample and preconsolidation pressure. In addition, test reports should include the plotted graphs discussed in the applicable ASTM standard that shows the various relationships between void ratio, applied stress, and coefficient of consolidation (c_v). General guidelines for reporting test results are provided in ASTM D2435 and D4186. Preconsolidation stress should be estimated using a recognized method (i.e., the Casagrande method) and documented in the laboratory test results. Levee designers should always evaluate the validity of the preconsolidation stress estimated from laboratory test results since even recognized methods include subjectivity in the estimation. Designers should justify the preconsolidation stress ultimately used in the design. If the site contains organic soils, or secondary compression is a factor be considered in design, the coefficient of secondary compression should be developed from the consolidation data.

3.3.3.1 Incremental Load Method (ASTM D2435). The test specimen should generally be loaded, in increasing increments, to loads that range from 0.0625 to 16.0 tons per square foot unless higher stress levels are needed due to soil stress history, depth of the sample, or height of levee. The final load in the test should be equal to or greater than four times the preconsolidation pressure of the test specimen. Readings of deformation (as assessed from deflection gage readings) versus time should be measured and recorded at the following times: 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 minutes and 1, 2, 4, 8, and 24 hours, or 100 percent primary consolidation. If primary consolidation has not occurred in the first 24 hours, hold the load for an additional 24 hours each day until primary consolidation has occurred. To confirm consistent material behavior over varying stress levels, constant reloading time increments must be used. After the maximum desired load has been achieved, the test specimen should be rebounded by removing the load in decrements. This normally is done by taking three-quarters of the load off successively for each decrement. Two or more decrements are required for a valid rebound curve.

3.3.3.2 Constant Rate-of-Strain (ASTM D4186). Constant rate-of-strain (CRS) method offers advantages over incremental load method as the CRS method is usually completed in shorter time than the incremental load method. However, disadvantages of the CRS method include: secondary compression data is not easily determined, the strain rate established for the test requires careful consideration, and application of a rebound-reload hysteresis loop may slow down testing. Advantages and disadvantages of CRS consolidation tests should be weighed prior to specifying this test for levee projects. The test specimens should be loaded to a final load of 16 tons per square foot (unless higher stress levels are needed), using a constant rate-of-strain, as specified in ASTM D4186. Reduced maximum loads can be used if it is verified that the final load is greater than four times the preconsolidation pressure of the test specimen.

3.4 Soil Compaction. The type and number of compaction tests will be influenced by the method of construction and the variability of available borrow materials. Laboratory compaction tests are needed to assess the compaction characteristics (such as optimum moisture content and dry density) of the soil. These characteristics can be used to evaluate the sufficiency of the soil placement and compaction efforts during construction. Soil samples compacted to laboratory test standards can be tested to verify desired properties of strength, hydraulic conductivity, and density for design and construction activities. Compaction effort of compacted soils can be verified in the field with tests such as sand cone and nuclear gauge.

3.4.1 Standard Proctor Compaction Effort (ASTM D698) and Modified Proctor Compaction Effort (ASTM D1557). Generally, for levee embankments, laboratory compaction test ASTM D698, Laboratory Compaction Characteristic of Soil Using Standard Effort, is used. Levees embankment fills constructed using the results of ASTM D698 as the basis for compaction control typically meet the necessary strength, hydraulic conductivity, and density properties assumed in most designs. ASTM D1557, Laboratory Compaction Characteristics of Soil Using Modified Effort, is seldom used for levees where fine-grained soils are used as fill, although modern compaction equipment is capable of compaction to this higher reference standard and this test can also be used for levee construction. The use of the ASTM D1557 may be more appropriate where more coarse-grained materials are used for levee construction or roadways atop levees, higher shear strength is required for stability, lower hydraulic conductivity is required, and settlement of fill materials needs to be minimized. When selecting a higher compaction effort for design and construction control, designers should be aware that strain incompatibility can occur when highly compacted, strong fill materials are constructed over soft foundation soils. Strain incompatibility may result in cracking of embankment materials, as materials compacted to a much higher strength than the foundation may be more brittle when constructed over a highly compressible, softer foundation.

3.4.2 Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table (ASTM D4253) and Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density (ASTM D4254). Compaction of pervious, free-draining fill materials with fines content less than 15 percent can be controlled using relative density and the maximum and minimum density test results from ASTM D4253 and ASTM D4254, respectively. Materials that are not free-draining and have fines content greater than 15 percent should use one of the impact compaction tests (ASTM D698 or ASTM D1557) to determine the moisture-density relationship and should use relative compaction specifications. It has been well documented that reproducibility of both ASTM D4253 and ASTM D4254 can be difficult between different laboratories, which can make the use of these test results for relative density a difficult specification to enforce. Thus, it may be preferable to use relative compaction specifications for cohesionless fill where the maximum density is determined from impact compaction tests (ASTM D698 or ASTM D1557), the vibratory table (ASTM D4253), or a vibrating hammer (ASTM D7382). Soils that have a fines content between 5 and 15 percent should be tested with a combination of impact compaction tests and vibratory compaction tests to evaluate the appropriate test to use for control during construction. The maximum density of soils with a fines content less than 5 percent is best determined with one of the vibratory compaction tests (ASTM D4253 or ASTM D7382). Ultimately, it is the responsibility of the designer to determine what type of compaction control (relative density or relative compaction) and what types of laboratory tests to specify considering factors such as (but not limited to): site-specific materials, availability of testing equipment, past experience in the region, and local practice to control the degree of compaction of cohesionless material. Refer to Chapter 10 for details on soil compaction testing during construction.

3.5 Soil Shear Strength. Results from soil shear strength testing should be commensurate with the conditions intended for their use. The laboratory testing program should consider the need for both effective stress and total stress (undrained) shear strengths. Discussion on slope stability conditions and required shear strengths are provided in Chapter 7. Shear strengths of proposed levee embankment materials may be completed on laboratory compacted samples from borrow pit materials during the design phase. For evaluation of existing, fine-grained embankment materials and levee foundations, laboratory testing should be completed on undisturbed samples. For determination of shear strengths of cohesionless soils (i.e., undisturbed samples cannot be obtained), correlations with SPT or CPT penetration resistance should generally be used, rather than using shear strength tests using re-constituted laboratory specimens. Construction control of new levee fill typically does not require post-construction shear strength testing, although a few record tests may be performed. The quality of the tests results are dependent upon the quality and care exercised during sampling and testing. For native materials, generally only undisturbed samples should be used for shear strength testing. For levee embankment materials, compacted samples are used for shear strength testing. Common soil shear strength tests include unconfined compression (UC), triaxial compression, direct shear, and laboratory miniature vane shear tests. Less common tests (such as triaxial extension) can be difficult to implement in a design and construction setting due to lack of experienced and qualified testing laboratories. Many levees have been designed and constructed with common shear strength tests. Thus, the engineering experience and historical performance of levees are related to these tests. The use of less common tests may be needed for some typically high cost or high risk projects, but a full understanding of the tests is needed for proper implementation in design and construction activities.

3.5.1 Unconfined Compression Tests (ASTM D2166). Unconfined compression tests (UCT or UC) can sometimes be used to assess the undrained shear strength of low-permeability soils. The UCT is essentially an unconsolidated-undrained triaxial compression test (UU) without any confining pressure. The UCT is more of an index test than a true shear strength test due to the lack of confining pressure applied to the sample. The UCT should not be used as the primary determination of undrained shear strength when undrained shear strength is a controlling factor in the levee design. Under certain specific conditions, such as for some saturated, normally consolidated clays not vulnerable to de-gassing during sampling, unconfined compression tests may be performed on samples that are homogeneous (i.e., no sand lenses) and without joints or slickensides to provide a rough assessment of undrained in-situ strengths. Unconfined compression tests are simpler to perform than triaxial compression tests, but test results generally exhibit more scatter. For over-consolidated materials, undrained strength may be many times greater than drained strength and UCT might not be the appropriate measure for shear strength in analysis. UCT specimens generally have a diameter of equal or greater than 1.4 inches and a minimum length of 2 times the diameter. UCT specimens should be sheared at a rate of 1.0% per minute for plastic materials, or 0.5% per minute for brittle materials that achieve a maximum deviator stress at 3 to 6% strain. When it is specified to run UC and UU tests on similar materials, it is best to shear the UC tests at 1% per minute for comparison to UU test data. It is important that the loading apparatus and instrumentation possess the necessary vertical force resolution to accurately measure the unconfined compressive strength. For soft soils, this may require load cell capacities of 100 pounds or less. Recorded UCT results should include but not be limited to boring name, sample visual description, sample elevation, sample location, strain rate, specific gravity, water content, wet density, dry density, saturation, void ratio, diameter, and height. The results of the UCT should be displayed with compressive stress versus axial strain plots, to include unconfined compressive strength, failure strain, estimated undrained shear strength, and sketch or photograph of the failure plane.

3.5.2 Triaxial Compression.

3.5.2.1 Types of triaxial compression tests include unconsolidated-undrained test (UU) (ASTM D2850), consolidated-undrained (CU) (ASTM D4767), and consolidated-drained (CD) (ASTM D7181). The UU, CU, and CD triaxial tests have in the past been referred to as Q, R or R-bar, and S tests, respectively. The CU triaxial test should include pore pressure measurements taken during shearing. CD triaxial tests typically do not require pore pressure measurements except to verify back pressure saturation. CU and CD triaxial tests may be consolidated under isotropic or anisotropic conditions depending on structure loading conditions and associated testing requirements. The UU test is commonly used to assess undrained shear strengths for slope stability analysis. CU tests are less commonly used to assess undrained shear strengths for rapid drawdown and seismic stability analyses, and are more commonly used to develop effective stress shear strengths. CD tests may be used for assessing drained shear strengths in a slope stability analysis, though care must be taken to ensure that strain rates are slow enough to ensure 100% drainage during the test. Generally, the tests are conducted on 3-inch or 5-inch diameter samples that are cut into equal specimens such that each specimen can be trimmed for testing. The specimen size for triaxial testing should generally be 1.4 inches or larger in diameter and 2 to 2.5 times the diameter in length. For CU tests, failure envelope is typically evaluated by a suite of at least three tests performed at three different consolidation stresses. The fourth specimen (from 5-inch diameter samples) can be tested if verification of one of the first three tests is necessary. Generally, the maximum confining pressure shall exceed the maximum effective normal stress expected in the field in order that the effective stress along the failure plane of the lab sample will be at least that expected in the field. The selection of confining pressures for strength testing should carefully consider the existing, past, and future stress conditions that will develop over the range of anticipated stress loadings and unloadings. Care should be taken not to rely solely on the results of UU and CU tests completed on samples that are significantly impacted by sample disturbance. Other methods, such as in-situ testing and normalized shear strength parameters (i.e., such as those obtained from SHANSEP), should be employed where reliance on UU and CU test results may be unreliable due to sample disturbance. Piston, membrane, and filter paper corrections must be applied, where applicable, to test results. The axial load induced to the specimens should generally be performed at a rate as follows:

- UU triaxial tests should generally be sheared at a rate of 1.0% per minute for plastic materials, or 0.3% per minute for brittle materials that achieve a maximum deviator stress at 3 to 6% strain.

- CU triaxial tests should be completed with pore pressure readings to allow for development of both drained and undrained shear strengths parameters and should generally be sheared at a maximum rate of about 0.1% per minute or reference guidelines in ASTM D4767 for plastic materials (at least 120 minutes to maximum deviator stress). If failure is expected to occur after 4% strain, a suitable strain rate is determined by dividing 4% by ten times the value of t_{50}.

3.5.2.2 It is important that the loading apparatus and instrumentation possess the necessary resolution to accurately measure the shear forces and deviatoric forces. For soft soils, this may require load cell capacities of 100 pounds or less. CU tests require a satisfactory process in the laboratory for obtaining proper saturation. Saturation cannot be achieved unless the laboratory carefully and incrementally brings the samples up to a minimum of 50 pounds per square inch (psi) backpressure to achieve saturation. B-value verification of saturation should be properly documented. Recorded results from triaxial tests should include but not be limited to the boring name, sample visual description, sample elevation, sample location, grain size, Atterberg Limits, unit weight, specific gravity, water content before and after shear, dry density, saturation, void ratio, diameter and height, back pressure, cell pressure, failure stress, ultimate stress, deviator stress at failure, and a sketch or photograph of the failure plane. In addition, plotted stress strain curves and Mohr's circle plots should be furnished for each specimen tested. ASTM defines failure as the peak deviator stress if the peak occurs at an axial strain of 15% or less. If the peak occurs at a higher strain, then failure is defined as the deviator stress at an axial strain of 15%. ASTM standards also allow for the principal stress ratio to be used to define failure. Mohr's circles should be plotted for the test specimens at failure. Guidelines for reporting can also be found in the referenced ASTM standards. It is generally preferable to have a companion consolidation test to assess the stress history of the sample prior to testing, allowing for more refined assessments of appropriate triaxial testing consolidation stresses.

3.5.3 Direct Shear (ASTM D3080). The direct shear strength test is a CD test that may be used to obtain the drained shear strengths of relatively free-draining and some lower-permeability materials for slope stability analysis. In some situations, the direct shear test may be preferred over CD triaxial tests to evaluate the drained strength of clays, since they can be sheared faster than CD triaxial tests due to the shorter drainage path of the sample. Although direct shear tests can be used to obtain drained shear strengths, undrained conditions cannot be verified during testing. There is some uncertainty in the level of drainage that actually occurs during the test and this uncertainty should be considered when evaluating direct shear strength test results. The effective stress at failure for direct shear test specimens should bracket the anticipated normal effective stress on the failure plane in the field. Direct shear tests should be performed in accordance with ASTM D3080. Direct shear tests require loading rates where time of failure (t_f) = 50 * t_{50}. This may require a shear rate as low as 0.001 inch per minute. Other guidance for shear rates is found in ASTM D3080. Recorded results from direct shear tests should include but not be limited to the boring name, sample visual description, sample elevation, sample location, grain size, Atterberg Limits, unit weight, specific gravity, water content before and after shear, dry density, saturation, void ratio, specimen size (height, width, and length), normal stress, and failure shear stress. Guidelines for reporting can also be found in the referenced ASTM standards. It is important to note that the Direct Shear test was withdrawn by ASTM in 2020. There is currently no replacement test and it should continue to be used until a replacement test is approved by ASTM

3.5.4 Laboratory Miniature Vane Shear (ASTM D4648). Laboratory miniature vane shear tests may be used to assess the undrained shear strength of saturated, cohesive soil samples. In many respects, the undrained shear strengths obtained from this test are more of an index value that allows for a relative evaluation of undrained shear strength than an absolute value that directly represents it. Laboratory miniature vane shear tests should be conducted in accordance with ASTM D4648. The ASTM standard allows for the test to be used where undrained shear strengths are less than 2,000 pounds per square foot; however, the test is more commonly run on softer clays. This test should not be used in lieu of UCT or UU triaxial tests for soft saturated clays, but should rather be another tool which complements the overall laboratory and in-situ testing program.

3.5.5 Direct Simple Shear Test (ASTM D6528). Direct Simple Shear tests (DSS) tests may be used to assess the undrained shear strength. However, DSS tests are not commonly used in levee design and construction activities. DSS should be conducted in accordance with ASTM D6528. DSS test specimens should be consolidated to vertical effective stresses equal to or greater than the in-situ vertical effective stress for normally consolidated soils and should be consolidated to higher stress levels for overconsolidated soils. Test specimens consolidated to stresses less than the vertical effective stress will likely exhibit over-consolidated behavior and may not provide data directly applicable to the conditions being analyzed.

3.6 Hydraulic Conductivity.

3.6.1 Hydraulic conductivity determination of foundation and levee embankment material is necessary for performing seepage analysis discussed in Chapter 6. Hydraulic conductivity estimates can be made from various sources including published data, field tests, laboratory tests, and empirical correlations. When making a hydraulic conductivity determination for a particular application or analyses, it is important to understand the differences in macro-permeability and micro-permeability. The macro-permeability is the overall hydraulic conductivity of the soil horizon (which includes fissures, cracks, root holes, seams, animal burrows, etc.) and is representative of actual field conditions. Laboratory hydraulic conductivity test results are generally more reflective of the micro-permeability, which is the hydraulic conductivity of a small, discrete and relatively uniform sample that is not influenced by the larger defects in the soil matrix that are seen in the field. The macro-permeability for fine-grained material is often higher than indicated by the values obtained from laboratory hydraulic conductivity tests due to the discontinuities and imperfections in the soil horizon. Consequently, laboratory hydraulic conductivity tests of fine-grained fill materials or surface clays overlying pervious foundation deposits are not often performed as part of routine laboratory testing programs as the macro-permeability may be different than the hydraulic conductivity indicated by laboratory hydraulic conductivity tests by more than an order of magnitude.

3.6.2 Anisotropy of the soil can have a significant impact on the hydraulic conductivity determination due to the differences in the horizontal hydraulic conductivity and vertical hydraulic conductivity. Anisotropy of the soil is primarily due to the method of deposition or placement but also can be influenced by particle shape and orientation. Chapter 6 discusses the general flow regime through and beneath levees.

3.6.3 Macro-permeability estimates may be desired in a seepage analysis and can be determined from field tests or site specific seepage data (including piezometric pressures and seepage flow). ASTM D4043 offers a standard guide for selection of aquifer test methods (field test). When performing field tests to evaluate hydraulic conductivity, it is important to understand that different methods may allow for an estimate of horizontal hydraulic conductivity, vertical hydraulic conductivity, or more of an average hydraulic conductivity that combines horizontal and vertical hydraulic conductivity.

3.6.4 TM 3-424 (1956), pages 255-266, provides vertical hydraulic conductivity of natural blankets along the lower Mississippi River based on seepage data collected at 16 sites. TM 3-424 notes that the macro-permeability of natural blankets is related to the thickness of the blanket and is influenced by the presence of defects (root holes, shrinkage cracks, minute fissures, and animal burrows). For thin blankets (less than 10 feet thick), the seepage flow through the blanket was noted to occur predominantly through the defects and not the soil matrix.

3.6.5 Hydraulic conductivity determination from laboratory tests can be made from Constant Head Permeability tests (ASTM D2434), Flexible Wall Permeameter (ASTM D5084) and consolidation tests Incremental Load Method (ASTM D2435) or the constant rate-of-strain method (ASTM D4186). Constant Head Permeability tests are limited to disturbed coarse-grained soils containing not more than 10% soil passing the 75- µm (No. 200) sieve. The Flexible Wall Permeameter test is the most common permeability test for fine-grained soils and is typically limited to soils with a hydraulic conductivity less than about 1×10^{-4} cm/s. Hydraulic conductivity determination from consolidation tests is better suited for fine-grained soils. It is important to note that these laboratory tests measure the vertical, micro-permeability of the soil.

3.6.6 Laboratory hydraulic conductivity tests on natural blankets are not often performed as a part of routine laboratory testing, because they often underestimate the actual macro-permeability. The designer should assess the need for laboratory hydraulic conductivity tests on natural blanket materials based on local experience, geologic conditions, and project requirements.

3.6.7 Laboratory hydraulic conductivity tests on natural coarse-grained deposits (i.e., sands and gravels) are rarely performed due to the difficulty and expense of obtaining undisturbed samples for testing. Hydraulic conductivity determination of natural coarse-grained deposits can also be made from field tests and correlations based on grain-size distribution (if applicable to the site-specific soils). Correlations based on grain-size distribution often provide an empirical relationship of the horizontal hydraulic conductivity versus various properties of the soil (i.e., D_{10}, void ratio, density, etc.) (e.g., California Department of Water Resources Urban Levee Evaluation Project Correlation provided in Chapter 2). Correlations are often based on limited testing, range of grain-size distribution, soil particle shape, and soil particle size. The designer should exercise caution when using correlations to estimate permeabilities and assess the impact of the hydraulic conductivity estimate relative to the level of reliability required.

3.6.8 TM 3-424 (1956), Figure 17, provides an empirical relationship of the horizontal permeability and d_{10} for lower Mississippi River alluvium sands. This relation is based on numerous pump tests conducted along the lower Mississippi River.

3.6.9 Several other correlations based on grain size distribution exist including Hazen's Method (Hazen 1892, 1911) and Kozeny-Carman equation (Kozeny 1927; Carman 1938, 1956). Hazen's Method is applicable to loose sands with a d_{10} between 0.1 mm and 3 mm. Hazen's Method is considered to be a fairly crude estimate of hydraulic conductivity that can be heavily influenced by the assumed Hazen coefficient (Carrier 2003). The Kozeny-Carman equation can be used to estimate the hydraulic conductivity for a wide range of soils including clean sands to non-plastic silts. Kozeny-Carman correlation considers the void ratio and angularity of the particles as well as the entire particle-size distribution of the soil. Guidance on how to develop input parameters for the Kozeny-Carman equation and how to apply the equation are included in Carrier (2003) and in Chapuis and Aubertin (2003b).

3.6.10 Published hydraulic conductivity data of compacted soils is provided in USBR (2014). This data includes vertical hydraulic conductivity and hydraulic conductivity anisotropy (k_H/k_V) of a variety of soils used in embankment dams. Hydraulic conductivity data of compacted soils is also included in the 2010 Urban Levee Evaluations study (ULE) by the California Department of Water Resources (URS 2015).

3.7 Dispersive Soil Tests. Dispersive soils (often soils with high sodium content) are very susceptible to external erosion induced by overtopping or wave action and internal erosion due to seepage. If levee fill materials or potential borrow source is suspected to be dispersive (i.e., soils deposited in a geologic environment prone to dispersive soils, areas with low plasticity silty clay or silt soils, observations of sinkholes or depressions after rainfall events, etc.), dispersive soil tests such as crumb test, double hydrometer test, and pinhole erosion tests should be performed. For new fill material, dispersive clays should be avoided. If levee fill material contains dispersive clays, considerations should be given to protect dispersive soils exposed to seepage, overtopping, or wave action.

3.7.1 Crumb Test (ASTM D6572). The crumb test is a simple and quick method for field or laboratory identification of dispersive clay soils. The crumb test results, which indicate dispersive characteristics, are generally accurate but results which indicate non-dispersive characteristics may not always be accurate. Thus, if dispersive soils are suspected and not verified through the crumb test, other tests such as double hydrometer or pinhole erosion test may be necessary to complete verification.

3.7.2 Double Hydrometer Test (ASTM D4221). Double hydrometer test is another method of identifying dispersive soils. This test is applicable to soils with a plasticity index greater than 4. This test may not identify all dispersive clays, and other tests should be used in conjunction.

3.7.3 Pinhole Erosion Test (ASTM D4647). The pinhole test is a method of identifying dispersive characteristics of clays. The pinhole erosion test models the action of flowing water along a crack in an earth embankment. This test is considered a direct measurement of the dispersibility of clay soils, and results have generally correlated well with field performance.

3.8 Surface Erosion Tests. Non-dispersive soils are also susceptible to erosion, and both dispersive and non-dispersive soils can also be evaluated using the applied hydraulic shear stress approach, requiring estimates of the critical shear stress and erosion rate parameters (see Chapter 9). Thus, erosion tests such as the erosion function test or wave overtopping simulator may be needed to assess the susceptibility to erosion for the potential overtopping and wave action conditions for the levee.

3.8.1 Erosion Function Apparatus and Jet Erosion Tests. Erosion rates as a function of flow velocity can be measured in the laboratory using one of several devices such as the Erosion Function Apparatus (EFA) (Briaud et al. 2001a and b) and the Jet Erosion Test (Hanson 1990). The original Jet apparatus has been modified into a smaller more portable version (Hanson and Cook 1999) for in-situ testing in open channel flow. Both the EFA and Jet Erosion test and analysis methods allow for estimation of soil erosion based on an applied shear stress at the water/soil interface, giving values for the shear stress necessary to just start erosion (i.e., the critical shear stress) and the erosion rate constant, k. The k constant is typically presented as a linear relationship between applied stress and rate of erosional losses, as represented by a depth of soil removed.

3.8.2 Wave Overtopping Simulator. In recent years, Wave Overtopping Simulators (WOSs) have been developed for evaluating wave, constant overflow, and mixed overtopping of levee erosion resistance. Some of the devices are permanent facilities with large supporting hydraulic systems used to test samples fabricated in test trays. Others are portable and have been installed on levees to simulate the flow of overtopping waves over the landside slope. While testing of real levees improves representativeness of results, the portable hydraulic systems may not be able to generate sufficient overtopping flow rates to test some more-resistant bare soil, grass, and man-made armored slopes to failure, which limits application of test results. Use of these simulators is not part of routine practice, but such testing could be considered to provide information that would improve the resiliency of high risk levees.

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CHAPTER 4

Borrow Areas

4.1 General.

4.1.1 Borrow areas are utilized to provide earthen material necessary for the levee project. Selection of suitable borrow areas requires compliance with engineering requirements as well as environmental, cultural, and water quality laws and regulations. In general, properly selected and designed borrow area(s) should satisfy the following:

- Borrow area(s) locations and configurations should not adversely impact the reliability (i.e., increase the potential for levee underseepage, instability, or erosion) of the levee project during and after construction
- Borrow area(s) contain suitable earthen material for the entire levee project
- Borrow area(s) are in compliance with local, state, and federal laws and regulations often related to environmental resources, cultural resources, and water quality
- Borrow area(s) are accessible (i.e., ingress and egress rights are not inundated by flood waters) during the construction of the levee project
- Borrow area(s) locations and configurations are optimized (to the extent practical, given requirements listed above) to minimize levee project construction costs

4.1.2 Environmental, cultural, and water quality requirements for borrow areas will vary based on local, state, and federal regulations, and are beyond the scope of this manual. This chapter focuses on the engineering requirements regarding the selection and design of borrow areas for levee projects.

4.2 Borrow Area Selection. The engineering requirements for selecting borrow areas include location relative to the levee, borrow area volume, availability of suitable borrow material, and natural water content of the borrow material for construction. These requirements and considerations are discussed below.

4.2.1 Borrow Area Locations.

4.2.1.1 Locating borrow areas directly adjacent to a levee is generally discouraged for modern design practice. Often, excavating adjacent to the levee, either on the landside or waterside, will shorten seepage paths and reduce resistance and increase flows, exacerbating the potential for failure from high pore pressures, internal erosion, or a combination of both. The failure modes and design analysis of the levee should include the layout, location, and excavation depths of borrow areas adjacent to the levee. Any proposed borrow area locations and configurations should meet the required reliability of the levee project (Chapter 1).

4.2.1.2 Generally, the most economical configuration is to establish borrow areas parallel and near to the levee; however, the trade-offs between ease, cost, and reduced reliability of the levee due to adjacent excavation must be carefully considered as discussed in Section 4.2.1.1. Long, shallow borrow areas along the levee alignment are economically more suitable because of the shorter haul distance involved. Large centralized borrow areas located offsite may be required for levees in urban areas or where adjacent borrow areas are unavailable.

4.2.1.3 Borrow area locations within the leveed area are generally more expensive and may create or exacerbate underseepage issues if placed close to the levee landside toe. Waterside borrow locations in some areas will be filled eventually by siltation, thereby negating the man-made changes in the landscape.

4.2.1.4 Channel or excavations required for the levee project features should be used to extent possible to limit the amount needed from designed borrow areas.

4.2.2 Borrow Area Volume.

4.2.2.1 Required borrow area volumes include levee cross-section volume, fill necessary for levee project features, and contingency volume. Borrow area volume contingencies are represented as a percentage of the levee cross-section volume and fill necessary for levee project features. Borrow area contingencies should account for borrow material losses, shrinkage of the borrow material, and levee settlement during construction. For most levee projects, borrow area volume contingencies range from 25% to 50% (i.e., borrow area volumes should be at least 125 to 150% of the levee cross-section volume and fill necessary for levee project features). Borrow area volume contingencies shown are based on levee construction experiences along the lower Mississippi River Valley (projects ranging from half a million to a million cubic yards of required earth fill) and careful consideration must be given for specific levee project conditions that may affect these contingencies. These considerations are provided below.

4.2.2.2 Borrow material losses occur due to a variety of factors including borrow material compaction methods, processing methods (i.e., drying of borrow materials), excavation and handling methods, rejection of unsuitable borrow materials, and extent of borrow area clearing and grubbing. Generally, more borrow material losses occur if significant processing of the borrow material is required, clearing and grubbing of trees in the borrow area is required, and/or borrow material must be handled multiple times. The volume of required earth fill for the levee project should also influence the contingency percentage used to account for borrow material losses. Small levee projects (i.e., volume of required earth fill is less than half a million cubic yards) typically require a higher contingency percentage for borrow material loss. This is due to higher material losses occurring during clearing and grubbing of the borrow area compared to the overall earth fill placed than for larger levee projects. Due to the numerous factors that can affect borrow material losses, typical ranges are impractical to report without considering these factors for the levee project and local experience in the methods utilized for the levee project.

4.2.2.3 Shrinkage of the borrow material is due to material compaction during placement. The amount of shrinkage can be estimated by comparing the natural material density to the expected material in-place density. Swelling of the borrow material can occur with rock or coral borrow material. Table 4-1 was adapted from U.S. Army Earthmoving Operations Field Manual No. 5-434 (Army 2000) can be used for preliminary volume conversions from borrow area to loose or compacted condition for a variety of soil material types. For semi-compacted or fully compacted fill, volume conversion from borrow area to compacted condition can be applied for preliminary estimates of borrow material shrinkage or swelling. A more accurate estimate of borrow area volume conversion should be based on the field dry density of the borrow material and the desired dry density of the compacted fill.

Table 4-1. Typical Volume Conversions from Borrow Area to Loose or Compacted Conditions. (adapted from U.S. Army FM No. 5-434 (Army 2000))

Soil Material Type	Converted from Borrow Area to Loose Condition	Converted from Borrow Area to Compacted Condition
SW, SP, GP, or GW	1.11	0.95
SM or ML	1.25	0.90
CL or CH	1.43	0.90
Rock (blasted)	1.50	1.30
Coral (comparable to lime rock)	1.50	1.30

Note: Values less than 1 indicate shrinkage and values greater than 1 indicate swelling.

4.2.2.4 Levee settlement during construction will depend on the height of the levee being constructed, compressibility of the levee foundation, rate of consolidation for the levee foundation, and duration of the levee project construction. For most levee projects founded on alluvial foundation soils, levee settlement during construction is very minimal compared to borrow material losses or shrinkage. However, for levee projects founded on highly compressible soils, levee settlement during construction can be significant and additional contingencies for borrow area volumes may be required. These contingencies should be based on the estimated settlement during construction.

4.2.3 Suitable Borrow Material.

4.2.3.1 Soil suitable for construction depends on the design of the levee embankment and required reliability of the levee. Generally, very wet, cemented, high plasticity, coarse-grained, or highly organic soils are not desirable for use in levee embankments and levee project features. However, the use of highly plastic clays is not prohibited, but should only be used when local practices have a demonstrated history of success and it is accounted for in design. A discussion on appropriate earthen materials for levee embankments and levee project features is provided in Chapter 10.

4.2.3.2 Borrow area material may not be homogeneous and may contain substantial pockets or layers of differing soils types (for example, layers of sands and silts in a clay borrow area). Mixing or blending of fine-grained with coarse-grained soils in order to obtain suitable borrow material is often not cost effective and can lead to undesirable and inconsistent material for the levee project. Thus, it is recommended to obtain borrow material that is suitable for the levee project in its natural state.

4.2.3.3 Soil stabilization (i.e., lime or cement stabilization) of the borrow material may be considered to obtain suitable material for the levee project. Appropriate types of soil stabilization can be found in Chapter 10. The use of borrow material requiring soil stabilization will depend on its cost effectiveness for the levee project or lack of suitable borrow material within a reasonable distance to the levee project. The performance during and after construction of stabilized borrow material should be considered prior to use.

4.2.3.4 A Phase I Environmental Site Assessment (ESA) should be performed to investigate the potential presence of hazardous, toxic, or radioactive waste (HTRW) in the vicinity of the proposed levee project and potential borrow areas. The Phase I ESA should be conducted in compliance with ASTM E1527, Standard Practice for Environmental Site Assessments: Phase I Environmental Site Assessment Process. The focus of the Phase I ESA should be to review existing and past historical information regarding the site. The Phase I ESA should document the history of the site to evaluate the potential presence of any HTRW in order to avoid any areas of concern. Borrow materials exhibiting hazardous waste characteristics (40 CFR 261.21 – 261.24: Characteristics of Hazardous Waste), even if naturally occurring, is not suitable borrow material and should not be used for the levee project.

4.2.4 Borrow Material Water Content.

4.2.4.1 For compacted levee earth fill, it is necessary to obtain borrow material with a water content to allow placement, adequate compaction, and adequate shear strength of the compacted material. Chapter 10 discusses moisture and compaction requirements for levee earth fill. Generally it is preferred to compact levee earth fill when it is wetter than the optimum moisture content as this results in lower hydraulic conductivities, higher erosion resistance, and more ductility to prevent cracking during settlement. Thus, borrow material with a natural water content that meets the moisture requirements for the levee earth fill is more cost effective and preferred. If the natural water content deviates from the moisture requirements, processing of the borrow material (i.e., drying, wetting, or soil stabilization) will be required and may deem the borrow material to be cost ineffective for use in the levee project.

4.2.4.2 Borrow area material typically undergoes seasonal variations in water content due to rainfall, flooding, or fluctuations in the groundwater. Therefore, evaluation of potential borrow areas should be based on samples obtained from borrow areas in that season of the year when levee project construction is planned to ensure that water content data used to determine if the borrow area is suitable for the levee project or if the borrow area material will require processing represents actual conditions that will be encountered on the levee.

4.2.4.3 The cost of drying borrow material to a suitable water content can be very high, in many cases exceeding the cost of obtaining material that can be placed without drying but must be hauled a longer distance. For waterside borrow areas, the time required to excavate or process the material may not be available during the non-flood season. If processing of the borrow material (drying, wetting, or soil stabilization) is required, it is generally recommended that the processing not be performed in the location where the fill will be placed (i.e., not on the levee embankment) to avoid increasing the moisture content of the lower lift and subsequently causing softening of the lower lift. Processing areas for borrow material should be accounted for in the borrow area design or levee project layout.

4.3 Borrow Area Design. The borrow area design includes the layout of the borrow area, depths, slopes, surface drainage, flow conditions, and environmental design considerations. In general, these are standard requirements for the borrow areas. Additional requirements for mitigation of impacts to environmental resources, cultural resources, and water quality should be considered and incorporated into the borrow area design as required.

4.3.1 Borrow Area General Layout.

4.3.1.1 It is generally preferable to have borrow areas “wide and shallow” as opposed to “narrow and deep.” While this may result in additional environmental resource impacts and require extra right-of-way and a longer haul distance, the benefits derived from improved levee stability and underseepage conditions and long term environmental conditions usually outweigh the extra cost. For waterside borrow areas, “wide and shallow” rather than “narrow and deep” borrow areas also help prevent river channel migration or scour adjacent to the levee and promote material disposition in the borrow area during high water events. Right-of-way requirements during active use of the borrow pit should be established typically 15 to 20 feet beyond the top of the planned outer slope of the borrow area. This extra right-of-way will allow for maintenance of the borrow slopes, and can provide borrow material for levee maintenance if needed later.

4.3.1.2 As shown in Figure 4-1, a natural berm should be left in place between the levee toe and the near edge of the borrow area. The berm width depends primarily on foundation conditions, levee height, and amount of land available. Its width should be established by seepage analyses where pervious foundation material is close to the bottom of the borrow area and by stability analyses where the excavation slope is near the levee. Minimum berm widths used frequently in the past are 40 feet waterside and 100 feet landside which are based on ease of levee construction operations and to prevent adverse impacts to the reliability of the levee. However, natural berm widths should be wide enough to achieve the required levee reliability and necessary to mitigate associated levee failure modes. Generally berms should be included in the operations and maintenance corridor surrounding the levee except in cases where reliability analysis of the levee indicates otherwise.

4.3.2 Foreshore. The foreshore is the unexcavated zone between the waterside edge of the borrow area and the riverbank as shown in Figure 4-1. A foreshore width of 200 feet or more has been used to prevent migration of the river channel into the borrow area. Wider foreshore widths may be required to accommodate bridge foundations and/or buried utilities located waterside of the levee and ensure these features are not impacted by river channel migration and scouring. A waterside borrow area should not be used if the required foreshore width is not possible due to proximity of the riverbank.

4.3.3 Traverse. A traverse is an unexcavated zone left in place at intervals across the borrow area as shown in Figure 4-1. Traverses provide roadways across the borrow area and provide foundations for transmission towers and utility lines. For waterside borrow areas, traverses prevent less than bank-full river flows from coursing unchecked through the borrow area and encourage material deposition in the borrow area during high water. Experience has shown that when traverses are overtopped or breached, severe scour damage can result unless proper measures are taken in their design. The height of a traverse above the bottom of the borrow areas should be kept as low as possible when it will be used primarily as a haul road. In all cases, flat traverse slopes (on the order of 1V:6H to 1V:10H) should be specified to minimize scour caused by overtopping. If the traverse carries a utility line or a public road, even flatter slopes and possibly stone protection should be considered.

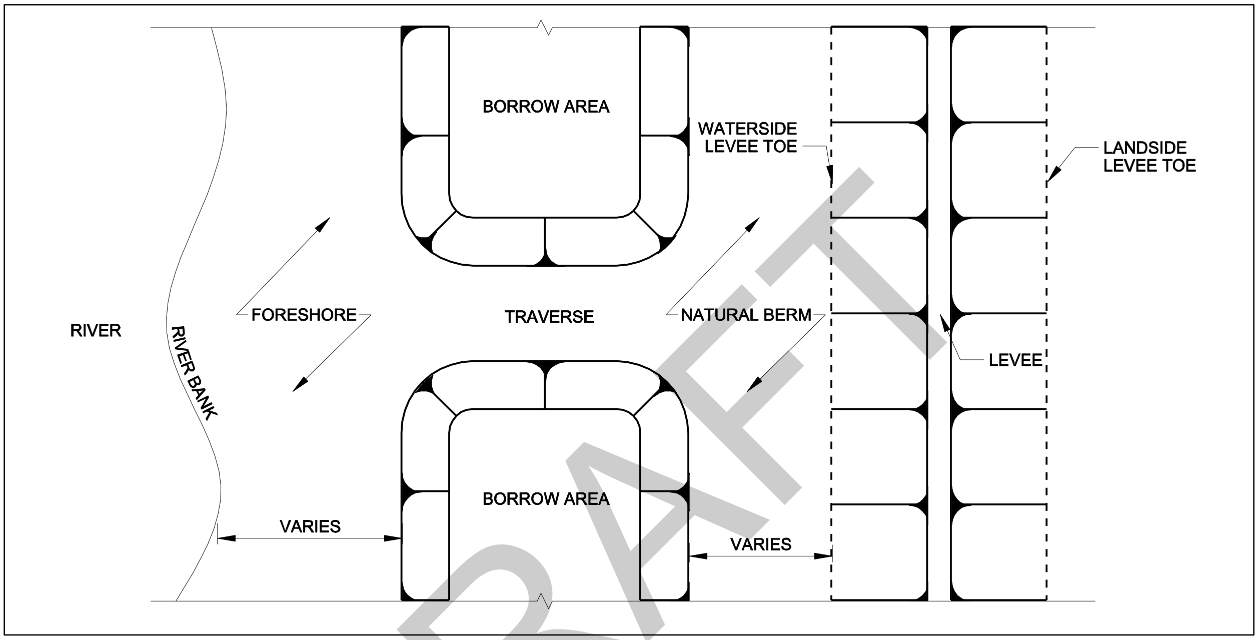

Figure 4-1. Borrow area layout showing typical levee and waterside borrow areas with traverse and foreshore. For landside borrow areas, this general layout also applies.

4.3.4 Slopes. Permanent excavation slopes of borrow areas should be designed for adequate stability as specified in EM 1110-2-1902. For borrow area slopes adjacent to the levee, the impacts to levee stability should be assessed using information in Chapter 7 and borrow area slopes should be designed to meet the required levee reliability. If borrow area slopes are intended to be maintained and mowed, a 1V:3H or flatter slope should be used. Where landside borrow areas are to be placed back into cultivation, changes in grade must be gentle enough to allow farm equipment to operate safely. If waterside borrow areas are subjected to river flows at high water stages, the slopes of the upstream and downstream ends of waterside borrow areas should be flat (i.e., on the order of 1V:6H to 1V:10H) to avoid erosion. For borrow area slopes that consist of pervious materials and subject to ground water exiting the slope, slopes 1V:5H or flatter may be required for a stable slope.

4.3.5 Clearing, Grubbing, and Stripping. Borrow areas should be cleared and grubbed to the extent needed to obtain fill material free of objectionable matter (such as trees, brush, vegetation, stumps, and roots). Subareas within borrow areas may be specified to remain untouched to preserve standing trees and existing vegetation. Topsoil with low vegetative cover may be stripped and stockpiled for later placement on outer landside slopes of levee embankments and seepage berms. Topsoil from borrow and levee foundation stripping can also be stockpiled and spread over the borrow area after borrow excavation has been completed. This reinforces the impervious cover and provides a good base for vegetative growth. In some situations, unsuitable borrow materials may be wasted in the borrow areas. However, impacts on the overall borrow area design should be considered when wasting unsuitable borrow material in borrow areas.

4.3.6 Borrow Area Depth.

4.3.6.1 Depths to which borrow areas are excavated will depend upon (1) groundwater elevation, (2) changes at depth to undesirable material, (3) preservation of adequate thickness of impervious blanket to prevent under-seepage, (4) environmental considerations (discussed later in this chapter), and (5) levee reliability assessment (discussed in Chapter 1).

4.3.6.2 Groundwater elevation should be below borrow area depths, preferably a minimum of 5 feet below, to prevent wet borrow area conditions. Excavating borrow material under water or wet conditions is generally discouraged. Excavating borrow material below the groundwater elevation can cause heave of the borrow area and groundwater seepage into the borrow area. Dewatering of borrow areas may be required to lower groundwater to a safe level to complete borrow excavation. Depth to groundwater should also be examined by either piezometer installation, subsurface investigations (i.e., borings or cone penetration tests), or review of available regional groundwater elevation records. Seasonal fluctuations in the groundwater regime (perched and/or unconfined) should be considered and potential impacts during construction mitigated.

4.3.6.3 In borrow area excavations, an adequate thickness of impervious cover should be left over underlying pervious material. For waterside borrow areas, a minimum of 3 feet of impervious cover should be left in place or constructed if removed. For landside borrow areas, the impervious cover thickness should be adequate to prevent the formation of unfiltered seepage exits under expected hydraulic heads.

4.3.7 Surface Drainage. Waterside borrow areas should be located and excavated such that they will fill slowly on a rising river and drain fully on a falling river. This may require drainage pipes or ditches be provided through traverses and foreshore areas as needed for proper drainage and avoidance of fish capture. This will minimize scour in the area when overbank river stages occur, promote the growth of vegetation, and encourage silting where reclamation is possible. The bottom of waterside areas should be sloped to drain away from the levee. Culvert pipes should be provided through traverses, and foreshore areas should be ditched through to the river as needed for proper drainage. Landside areas should be sloped to drain away from or parallel to the levee with ditches provided as necessary to outlet points. Gravity outlets or pump stations should be located to minimize lengths of surface drainage within the area.

4.3.8 Hydraulic Flow Conditions. To avoid damage from confined or restricted flow through the waterside borrow areas, obstructions or impediments to smooth and uniform flow should be removed if possible, or else protective measures must be taken. Waterside borrow areas should be made as uniform in width and grade as possible, avoiding abrupt changes.

Removal of obstructions that could cause concentrated flow includes degradation of old levee remnants and of narrow high-ground ridges beyond the borrow area, as well as removal of timber from traverses and from foreshore areas immediately adjacent to the borrow area. Obstructions to flow that cannot be removed include transmission towers, bridge piers, and other permanent structures near the levee. In such areas, stone protection should be provided for the levee or borrow area slopes if scour damage is considered probable.

4.3.9 Environmental Design Considerations. The treatment of borrow areas after excavation to satisfy aesthetic and environmental considerations has become standard practice. The extent of treatment will vary according to the type and location of a project. The long term positive and/or negative impacts on levee performance of any borrow area treatment should be intentionally evaluated during design, construction, and maintenance. Potential failure modes considering the potential positive and negative impacts of existing or future vegetation should be assessed during risk assessments completed as part of the evaluation and design process. Generally, projects near urban areas or where recreational areas are to be developed will require more treatment than those in sparsely populated agricultural areas. Minimum treatment should include proper drainage, topographic smoothing (i.e., grading), and the promotion of conditions conducive to vegetative growth in areas that will not negatively impact levee performance. Insofar as practicable, borrow areas should be planted to conform to the surrounding landscape and consistent with the hydrologic and hydraulic conditions used in levee design flood hazard evaluations. Stands of trees should be left remaining on landside borrow areas if at all possible, and excavation procedures should not leave holes, trenches, or abrupt slopes. Restoration of vegetative growth is important for both landside and waterside areas as it is not only pleasing aesthetically but may serve as protection against erosion. Willow trees can aid considerably in drying out boggy areas. Waterside areas should not be excavated so deep that restored grass cover will be drowned out by long submergence (see Section 4.3.7 Surface Drainage). It is desirable that waterside borrow areas be filled in by natural processes, and frequent cultivation of these areas should be discouraged or prohibited, if possible, until this has been achieved. Guidelines for landscape planting for borrow areas adjacent to the levee are given in EP 1110-2-18. Those responsible for maintenance of completed levees should be encouraged to plant and maintain vegetation, including timber, in the borrow areas if it is determined that future vegetation growth will not negatively impact levee performance. There is a documented case where timber and thick vegetation in old borrow areas focused flow between the levee and the wooded area that led to erosion of the riverside clay blanket along the toe which directly contributed to a levee breach. That case history highlights the need to carefully consider all potential positive and negative impacts of any vegetation within borrow areas.

CHAPTER 5

Subsurface Interpretation

5.1 General.

5.1.1 Subsurface interpretation is the process of integrating information from geology and geomorphology, past performance, subsurface explorations, water surface and groundwater elevations, geophysical survey, and other available information. The integrated subsurface information is used as the basis for levee evaluation, design, and construction. The purpose of subsurface interpretation is to characterize subsurface conditions consistent with the available data and the related analysis methods, tools, and goals. Subsurface interpretation, including development of interpretation profiles and cross-sections, should be conducted prior to any analysis or design and should be part of project documentation. This section presents objectives and approaches for developing the following:

- Plans and profiles
- Reaches and sub-reaches
- Material properties
- Analysis cross-sections

5.1.2 In general, data presentation (whether raw or processed) will be investigation-method-specific and should be presented such that subsequent engineering assessment could be performed with limited assumption on raw data. The subsurface interpretation tasks should place more emphasis on primary and more reliable data such as borings (e.g., standard penetration test (SPT) samples and/or "undisturbed" samples) with laboratory testing and less emphasis on those data which are dependent on assumptions such as cone penetration test (CPT) interpreted soil behavior type. However, CPTs are important for levee design because they provide a continuous record that provides more granularity in the soil profile and when calibrated to site specific conditions become a more reliable tool. Engineering judgment should be used to evaluate the reliability of the supplementary data (e.g., geophysical data) that may be dependent on investigation methods and site-specific conditions.

5.1.3 Examples of plans and profiles, reaches and sub-reaches, and cross-sections are provided in Appendix C. These examples are considered representative to illustrate discussions of this chapter. These examples are from the California Department of Water Resources Urban Levee Evaluations Project and recent levee design projects where USACE contributed as a partnering agency (URS 2015). Project-specific engineering judgment should be used in adopting or improvising these examples based on project objectives, site-specific conditions, and data availability. The examples shown in Appendix C include:

- Summary table for past performance
- Reach boundary map
- Legends for plans and profiles
- Plans and profiles
- Rationale table for reach and analysis cross-section selection

- Analysis cross-section figure
- Steady-state seepage and stability figures

5.1.4 Plan Development.

5.1.4.1 Plan view maps should be developed to provide an understanding of the levee and adjacent features, water bodies, rivers and/or streams, levee performance, surface geology, locations of explorations, and other site-specific features that impact evaluation of a levee. Plan views should be evaluated along with profiles in performing engineering evaluations of levees.

5.1.4.2 Plan views should show the following features and maps:

- Levee stations, levee miles, and river miles at regular intervals and at points of interest.
- Aerial maps showing levees, rivers and streams, and other physical features.
- Surficial geomorphologic maps showing geologic units with symbols.
- Exploration locations and types.
- Past performance distress areas (these may be isolated locations or areas along a stretch of levee) with dates.
- Locations of site-specific information (such as extents for geophysical study).
- Extents of existing or proposed mitigation measures.
- The scale of the map. The longitudinal scale should be same as that of the profile. The profile should be presented below the plan view. The transverse scale should be selected such that it adequately covers the areas of interest.
- Locations of utilities and other significant structures

5.1.4.3 Levee Stations, Levee Miles, and River Miles. Levee stationing, levee miles, and river miles should be identified at the beginning of a project. Levee mile (LM) and river mile (RM) systems usually exist for a levee system in levee operations and maintenance manuals or other documents. A levee stationing system should be developed along the levee alignment, which is usually along the levee centerline (e.g., levee crown). Correlations between project stationing, LMs, and RMs should be developed. LMs and RMs should be presented in plans and profiles at regular intervals (wider than station intervals) and at important intermediate locations, such as the boundaries between different sponsoring agencies, if applicable.

5.1.5 Profile Development.

5.1.5.1 A profile should be developed along the crown of the existing levee or the proposed levee crown alignment. If data collected on levee toes or other areas are available, they may be included on plan and profile drawings by projecting this data onto the profile (if appropriate to do so) or by developing elevation profiles at other appropriate locations such as along the levee toe.

5.1.5.2 Because profiles are often orders of magnitude longer than deep, it is common to portray them with an unequal aspect ratio (horizontal scale to vertical scale). Ratios of 10 times or more may be appropriate depending on the nature and amount of information available and the complexity of the subsurface stratigraphy. If unequal aspect ratios are used, it is recommended that the vertical scale on the profiles be the same as the vertical scale on interpretation cross-sections so that they may be readily overlain to confirm consistency in interpretation of subsurface features. Profiles should show the following features and maps:

- Design water surface elevations and other water surface elevations of interest to the project.
- Elevations of the landside and waterside levee toes and other areas of interest such as landside or waterside ditches and localized depressions.
- Exploration stick logs such as borings, CPT data, and vane shear testing results. Section 5.2 includes information that should be presented in stick logs. Exploration stick logs should be presented with levee stations, ground surface elevations, and offset from levee centerline or survey baseline.
- Location of penetrations (e.g., pipes), if known.
- Existing or proposed seepage control measures (cutoff walls, seepage berms, relief wells)

5.1.5.3 Borings. The following data from boring logs should be included in profiles:

- Graphical presentations (known as stick logs) of undisturbed borings and or SPT borings
- USCS material types shown on profiles and sections should match reflect final boring logs that have been updated and corrected from field identifications based on the results of confirmatory laboratory test results.
- SPT blow counts should be shown in profiles and labeled whether corrected or uncorrected. Additional correction factors such as overburden stress, rod length, sampler type, and borehole diameter should be utilized for subsequent engineering evaluations.
- Fines content, as measured in the laboratory using sieve analysis tests.
- Atterberg limits and moisture contents. Plasticity Index (PI) and corresponding moisture content should be used in reporting Atterberg limits test results.
- Visible organic materials such as roots.

5.1.5.4 Cone Penetration Tests. The following data should be included with the graphical presentation of CPT stick logs in profiles:

- Corrected total cone resistance (q_t) and friction ratio (R_f) in percentage, as calculated based on ASTM D5778.
- Porewater pressure generated immediately behind cone tip (u_2) superimposed on q_t plot. Availability of q_t, R_f, and u_2 will allow a reviewer access to the complete dataset that could be used for further engineering evaluation.
- Groundwater level measured from porewater dissipation testing.
- USCS soil classification, fines content, plasticity index, and moisture content of samples if collected during CPT testing.

5.1.5.5 Vane Shear Tests. The following data should be included along with the graphical presentation of vane shear logs (stick logs) in profiles:

- Peak and remolded undrained shear strength values from field vane shear tests, $(S_u)_{fv}$ and $(S_{ur})_{fv}$, respectively. These values should be presented as the adjusted values after the correction factor is applied.

- Plasticity index (PI) and sensitivity of soils (dimensionless).

5.1.5.6 Geophysical Explorations. Geophysical test result presentations should be testing-method-specific. As subsurface conditions in levee systems may vary significantly, manipulation of raw data using inversion techniques may not be appropriate for longer distances, or frequent inversions may be required. In the ULE study by the California Department of Water Resources (URS 2015), differential resistivity profiles were developed using raw resistivity data from an electro-magnetic (EM) survey with limited data manipulation. The resulting differential profiles were used to complement information from surficial geomorphic maps, exploratory borings and CPTs, and indications of past performance. Based on ULE experience, the use of EM surveys for levees can be considered a preferred method in areas where man-made obstructions such as utilities are not frequent. The profiles were used to interpret subsurface conditions with some success. An example of a differential resistivity profile is shown in Appendix C.

5.1.5.7 Appendix C presents recommended graphical and color standards to be used on boring logs, stick logs for profiles, and analysis cross-sections. The color scheme is based on relative hydraulic conductivity and is affected more significantly by the second letter in the USCS symbol than the first. The color of the USCS symbol is meant to spectrally mirror general values of hydraulic conductivity, with warmer colors (reds) being more permeable, cooler colors (blues) less permeable, and color differences proportional to hydraulic conductivity contrast. Yellow is assigned to clean sand and all other colors are relatively proportional to hydraulic conductivity.

5.1.5.8 Using these standard colors has several advantages. Choosing colors in an ordered spectral continuum allows analysts to identify approximate relative hydraulic conductivities between materials, leading to better predictions and quality control of seepage analysis results. Colors make interpretation easier and more salient than black and white symbols. Using standard colors when illustrating materials in slope stability and seepage analyses often makes the results easier to evaluate and helps in quality control and assurance reviews.

5.1.5.9 Engineering judgment should be used to decide whether site-specific continuous stratigraphic layers should be estimated and shown in profiles. Developing continuous stratigraphy of soil layers for long features such as levees becomes challenging if closely spaced exploration data are not available. If a sufficient number of explorations and other supporting information (e.g., geophysical studies and geomorphology maps) are available, a geologic profile with different geologic strata should be developed. However, in highly variable subsurface stratigraphy, a continuous geological stratigraphy may depict a conservative or un-conservative assessment of geologic conditions in absence of enough data. Because of difficulties in developing continuous stratigraphy in profiles, subsurface interpretation findings from plans and profiles can be documented in reach summary tables that list geologic information such as soil types, fines contents, Atterberg limits, and thicknesses of the blanket, aquifer, and aquiclude layers. Such a table with summary information should be used to develop rationale for developing a reach or sub-reach boundaries. An example of a reach selection rationale table is shown in Appendix C. In addition to a table, stratigraphy should be developed and drawn in analysis cross-sections as described in Section 5.6.

5.2 Reach Selection.

5.2.1 This section presents general guidance for selection of levee reaches for evaluation, design, and construction. These guidelines are informational and generalized. Engineering judgment should be applied considering site-specific conditions and data. The California Department of Water Resources has developed a reach selection protocol with input from architect-engineering (A-E) consulting firms, an Independent Consulting Board, and USACE for their ULE Project (2007 through 2012). These guidelines have been updated based on USACE experience and the updated guidelines are presented here.

5.2.2 A reach represents an extent of levee with similar conditions selected for evaluations and analyses and is represented by at least one analysis cross-section. Reaches, when combined, cover a levee segment.

5.2.3 A sub-reach is an extent of levee within a reach where conditions differ significantly. If these differing conditions are identified during the initial reach selection process, it would be appropriate to identify these as reaches. If these differing conditions are identified during subsequent data collection and evaluation, it would be appropriate to identify these extents as sub-reaches, thus avoiding re-identification of reaches.

5.2.4 It is not the goal, nor is it feasible, to identify, analyze, and evaluate every set of conditions existing along any particular levee. Reach selection makes use of a limited data set. When selecting reaches, analysts must consider interpretations and estimates of levee conditions and make interpretations and estimates that account for a range of conditions that, within reason, may exist in a levee and not just use conditions that are most likely or that may be more prevalent. Levees are essentially “chain-like” features whose performance is often dictated by the weakest, not necessarily the most representative, link. Interpreted reach boundaries should be conservatively extended or shortened based on the locations where differing levee conditions are confirmed.

5.2.5 Objective and Principles of Reach Selection.

5.2.5.1 Reach selection is the process of subdividing a levee system into discrete lengths such that each length has similar geotechnical, geometric, past performance, construction and remedial history, and hydraulic loading (differential hydraulic head above the landside levee toe). Reach selection enables incremental geotechnical analyses needed for overall levee evaluation.

5.2.5.2 The principal reach selection premise is that each reach or sub-reach should be represented adequately in terms of geotechnical characterization and analysis by at least one analysis cross-section and associated analysis parameters. Where conditions along the levee vary significantly enough that they are not adequately represented by the preceding reach, a new reach should be established.

5.2.5.3 In some reaches, one analysis cross-section may be adequate to represent different failure modes such as seepage, stability, rapid drawdown, erosion, and post-earthquake slope instability. In other cases, different cross-sections and evaluation conditions may be required to analyze failure modes within the same reach. In such cases, a set of sensitivity analyses on one analysis cross-section could be performed as an alternative to creating multiple analysis cross-sections.

5.2.5.4 The factors and characteristics that should be considered in establishing reaches are listed in Table 5-1.

5.2.6 Approach for Selecting Reaches. The approach for selecting reaches is to cover the study area with a reasonable number of reaches. As an example, the Natomas Levee Improvement Project in California had 48 reaches for 43 miles of Sacramento River and Natomas Cross Canal Levees. The Feather River West Levee Project in California had 42 reaches for 44 miles of levees. The following is guidance for dividing the levee into a reasonable number of reaches. The approach is based on a reasonable balance of the following factors and recommendations.

5.2.6.1 Analysts should generally characterize reaches by relatively consistent physical characteristics such that analysis results will apply over the entire reach.

Table 5-1. Factors to be Considered in Establishing Reaches.

Physical Features and Hydraulic Loadings	Past Performance and Maintenance Data
Crown width	Levee History (borrow source(s) and construction techniques)
Levee height (measured from landside levee toe to crown)	Historical performance such as underseepage and through-seepage, slope stability, erosion, etc.
Waterside slope conditions (waterside levee slope, bank slope, natural or man-made berm width and height)	Maintenance authority boundaries
Landside slope conditions (landside levee slope, localized steepness in slope)	Existing levee improvement measures such as levee raise and widening, cutoff walls, relief wells, seepage and stability berms, revetment, etc.
Topographic features such as landside ditches and localized depression areas	Recorded or estimated past high water elevations
Bathymetric data (river or stream channel bottom, erosional features, bends)	Animal burrowing control plan and existing conditions
Infrastructure encroachments and utility (bridges, pump stations, closure structures, utility crossings, pipes, etc.)	Inspection records and observations during floods (post-flood reconnaissance reports, levee logs, USACE periodic inspection reports, etc.)
Hydraulic loading (design, past high water events, authorized elevations)	
Land use (residential, agricultural, commercial)	
Real estate	
NEPA constraints (environmental, cultural)	
Geology/Geotechnical/Geophysical Data	
Regional geology	Geotechnical laboratory testing results
Geomorphology	Geophysical surveys
Levee embankment material type and borrow area	Historical investigations (by others) and quality of historical investigation data
Geotechnical explorations (borings, CPTs, vane shear, etc.)	Groundwater conditions

5.2.6.2 Developing additional reaches or sub-reaches may increase consistency in physical characteristics, but may not result in a change of analysis results. In such cases, analysts should not divide a single reach into two reaches if changes in physical and geological characteristics are not significant and the analysis results may not change.

5.2.6.3 The number of reaches may depend on project phases and data availability. The design phase of a project may have more reaches or sub-reaches than evaluation and feasibility phases based on availability of data and anticipated remedial measures.

5.2.6.4 Developing additional reaches or sub-reaches may not be supported by the amount of data available. Analysts should not divide a single reach into two reaches that lack significant characteristics to distinguish one from the other.

5.2.6.5 Reducing the number of reaches (or increasing reach lengths) may lead to conservatism or un-conservatism when characterizing some areas since a modeled cross-section may have to represent too broad an area, which could be difficult to capture in a single cross-section. Representing anomalous features in the same reach that do not coincide in the same cross-section should be avoided since doing so may result in overly conservative results. However, potential impact and extent of such anomalous areas should be evaluated and a sub-reach or another cross-section should be developed (See Section 5.4.8 for details).

5.2.6.6 Reaches fall into one of three descriptive categories: consistent reach conditions (either favorable or unfavorable), highly variable reach conditions, and anomalous or isolated reach conditions (usually a sub-reach). A consistent reach is one where conditions are relatively consistent over the entire length and a cross-section through any part of the reach would look similar to the modeled cross-section. Analysis results for these consistent reach conditions apply directly to the entire length of the reach. A highly variable reach is one where conditions are not consistent over the reach's entire length and where conditions vary over relatively short distances. A cross-section through any particular part of such a reach may look notably different from the modeled cross-section for that reach. Analysis might require generalized conservative assumptions for results to be valid for the entire reach. An anomalous reach is one that is substantially different in character from adjacent reaches, occurring most often as a sub-reach, where conditions over a limited length indicate additional analysis may be appropriate. The anomaly may be anything that changes expected performance such as a highly pervious aquifer underlying a thin blanket, a cutoff wall-improved length of foundation, or the presence of a special structure such as a pump-station and conduit.

5.2.7 Reach Selection Process. The reach selection process may be iterative with initial reaches selected based on stratigraphy, past performance, geometry, and/or hydraulic loading and final reach selection based on findings from analyses and further characterization. Most projects have several phases including evaluation, design, and construction. Reaches may be evaluated during any of these phases and may need to be modified to account for different analyses and findings. Reaches may differ between the phases depending on the objectives of the phases.

5.2.8 Identifying Sub-Reaches.

5.2.8.1 A levee sub-reach is a portion of the total length within a reach where conditions differ from the rest of the reach in a manner of particular interest. A sub-reach's conditions would not be accurately represented in a modeled cross-section of the rest of the reach. A sub-reach is relatively short in length compared to a reach. Examples of features that may warrant the identification of a sub-reach include the following:

- Topography and bathymetry differing from the rest of the reach, such as an isolated landside borrow pit at the levee toe.
- Levee geometry differing from the rest of the reach, such as an over-steepened slope where erosion has occurred around a pipe outfall.
- Levee construction history or remedial measures differing from the rest of the reach, such as a recently completed reconstructed levee or a relatively short cutoff wall.
- Historical performance differing from the rest of the reach, such as an isolated occurrence of slumping.
- A short length where geotechnical conditions differing from those in the rest of the reach, such as an isolated zone of marsh deposits.
- An encroachment on the levee such as a bridge crossing or pumping station.

5.2.8.2 To warrant a sub-reach, geotechnical conditions should be isolated and different than conditions elsewhere in the reach. Highly variable conditions throughout a reach may not warrant identification of a sub-reach. An example of a highly variable condition could be where a reach is mostly characterized by thick fine-grained foundation soils but includes small but spatially frequent channels of coarse-grained soils. For such conditions, it may be more appropriate to model both the typical and atypical conditions together within a single reach characterization.

5.3 Material Properties. For each reach, material properties should be developed for each stratum. Important properties for seepage and slope stability analyses include horizontal and vertical hydraulic conductivities and shear strength parameters. Challenges may arise when selecting parameters such as where unfavorable data in an area reflects inaccurate data or correctly indicates a poor condition in the field. These challenges may be resolved by considering both the reliability of the data and the potential consequence of being wrong.

5.4 Development of Analysis Cross-Sections.

5.4.1 The principal reach selection premise is that each reach and sub-reach should be represented adequately (in terms of geotechnical characterization and analysis) by at least one analysis cross-section and associated material properties. The following are several considerations when selecting analysis cross-sections.

- An analysis cross-section should be representative of the geology and geomorphology of the levee reach.
- An analysis cross-section should represent the most critical condition of a reach based on available data and failure modes.
- Levee geometry features such as a narrow crown width and steep slopes may indicate a short seepage path and potential for slope instability.
- Hydraulic head should be representative of reach conditions. For example, a location with a locally high levee toe elevation should not be selected as a representative analysis cross-section.

- Aquifer characteristics such as thickness, material type, continuity, and lateral extent should be considered.
- Blanket characteristics such as thickness, material type, continuity, and lateral extent should be considered. A thin blanket in a high hydraulic head condition may indicate potential for underseepage.
- Topography and bathymetry may indicate a direct connection between the channel and aquifer in the case of a river or stream channel.
- The presence and erodibility of a waterside blanket.
- Landside depressions and ditches should be considered critical as they may indicate thin blanket and high hydraulic head conditions.
- Soft or loose soil layers in the levee or foundation may indicate potential for slope instability during high water events.
- Erodible embankment materials may indicate potential for through-seepage, slope instability, and erosion. A sand lens within an embankment that daylights on the landside slope may indicate through-seepage potential.
- Utility crossings, if not properly designed and constructed, may be susceptible to piping.
- Flood walls should be evaluated considering soil-structure interaction effects.
- Past performance history may indicate one or more types of performance.
- Three-dimensional effects due to river bends, groin areas near a bridge abutment, or end of an existing or proposed mitigation area (e.g., cutoff wall and berm) may be present.
- The presence of multiple geologic units in a reach may warrant a comparison of subsurface data with geologic units when selecting the analysis cross-section.

5.4.2 The analysis cross-sections are generally taken transverse to levee alignment along the shortest seepage path. However, they could be taken at different angles if site-specific conditions require. Exploration borings may not fall on the analysis cross-section. In such cases, explorations from the vicinity could be considered and projected onto the analysis cross-section. The offset distance from the exploration location to the analysis cross-section should be shown. Sometimes analysis cross sections may be theoretical to reflect expected changed conditions. For example, one may develop an analysis cross section that presents the expected condition of channel meandering.

5.5 Interpretation and Analysis. Subsurface interpretation includes gathering pertinent data, developing graphical plan and profile presentations of the site, selecting reaches (and possibly sub-reaches), selecting design parameters for the soils in each reach, and developing representative cross-sections for the reaches. Once this subsurface interpretation is complete, geotechnical engineering analyses including seepage, slope stability, and settlement analyses can be performed and the results presented (see Chapters 6, 7, and 8). These results should be compared to the interpretation, history, and improvements that have been compiled in the graphical profile, plan, and cross-section drawings. As necessary, iterations of subsurface interpretation, strata interpretations, reach boundaries, and material properties may be needed where the initial analysis results do not compare well with levee performance history or expected results. Engineering judgment should be used when comparing initial analyses with past performance. Discrepancies between past performance and analysis results may be attributed to several factors. For example, the design flood elevations could be higher than in the past actual flood events, the duration of the events could be different, and/or conditions could have changed over time. When subsurface interpretations indicate significant variability, then designers should perform parametric analysis to reflect the range of conditions and the results of this analysis should be considered during Phase 2 (evaluate and adjust design using a risk assessment) of the evaluation and design process.

CHAPTER 6

Seepage Evaluation and Control

6.1 Introduction. This chapter is intended to be used to evaluate levee performance due to seepage during flood loading and to design levee seepage control measures to ensure the desired reliability of levee is achieved. This chapter covers analytical methods and criteria for both levee through-seepage (i.e., seepage through the embankment) and levee underseepage (i.e., seepage within the levee foundation) as well as general evaluation of seepage related failure modes. Although this manual is primarily written for levee embankments, analytical methods, criteria, and seepage control measures for levee underseepage are applicable to any levee feature unless otherwise noted in other levee feature specific guidance. Levee through-seepage analytical methods, criteria, and seepage control measures presented here are mainly applicable to levee embankments.

6.1.1 Levee performance due to through-seepage and/or underseepage is typically evaluated for most levees. The difference in water surface elevations on the waterside and landside of the levee and the associated seepage can cause levee performance issues. Seepage through or under the levee may produce water flow that exits at some point on the landside of the levee, which may require collection with filters and drains, and ultimately be conveyed and discharged with interior drainage. The movement of water under or through a levee can result in poor levee performance and/or failure modes (i.e., breach). Examples of poor levee performance are shown in the figure below. Seepage controls may be employed to ensure the reliability of the levee, which are discussed later in this chapter.

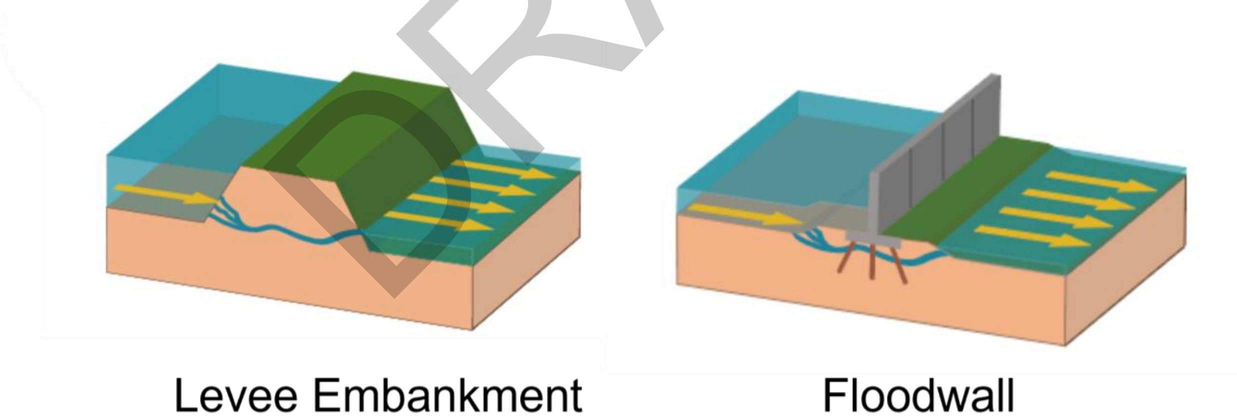

Figure 6-1. Example of Levee Perforamnce Issues Due to Seepage.

6.1.2 This chapter is divided into four sections as follows:

- Section I – Provides guidance on evaluation, analysis methods and deterministic design criteria for levee seepage.

- Section II – Provides guidance on final evaluation and design including internal erosion potential failure modes evaluation.

- Section III – Provides guidance on measures to control underseepage.

- Section IV – Provides guidance on measures to control seepage through levee embankments.

Section I
Levee Seepage Deterministic Evaluation and Design

6.2 General.

6.2.1 Traditionally, the approach to assess levee seepage has been to separately evaluate underseepage and through-seepage. However, with the increasing use and development of commercially available finite element method (FEM) and finite difference (FD) computer programs, both can now be evaluated simultaneously. For all types of levee seepage, the goal is to reliably prevent uncontrolled seepage and the resulting movement of either embankment or foundation materials by internal erosion or slope instability triggered by high pore pressure.

6.2.2 Both traditional and modern tools available for underseepage and through-seepage analyses require that in-situ soils and placed materials be characterized by discrete regions of homogeneous media with uniform engineering properties. Unforeseen soil deposits, cracks, or other defects in the assumed levee and foundation profile have the potential to result in drastically different performance in the field than anticipated in analysis and design. Because of this, practitioners understand analyses do not capture all seepage conditions or internal erosion failure mechanisms. Especially when evaluating existing systems that have performance history, the analyst may have to iterate on subsurface characterization, material properties, and seepage analysis parameters to get agreement between analysis results and field performance.

6.3 Underseepage Considerations.

6.3.1 Common levee underseepage conditions that require evaluation include:

a) Underseepage causing excess head at the bottom of a blanket overlying an aquifer. This condition can result in seepage, pin boils, or sand boils and the initiation of backward erosion piping (discussed later in this chapter).

b) Underseepage associated with relatively unrestricted flow through surficial coarse-grained materials. This condition is generally referred to as a ‘no-blanket condition’.

c) Underseepage within the bedding and backfill along and/or adjacent to utilities or penetrations such as conduits leading to internal erosion. Refer to EM 1110-2-2902 for guidance on evaluation and seepage control.

d) Underseepage along the interface with structures such as pump stations, floodwalls, roadways, bridges, and control structures leading to internal erosion.

e) Underseepage around a seepage control measure (such as cutoff wall and seepage berms), if not properly designed, leading to internal erosion.

f) Underseepage due to localized weakness in the foundation or due to animal burrowing and tree roots leading to internal erosion.

6.3.2 The configuration of a semi-pervious top stratum or blanket overlying a more pervious aquifer is relatively common in many US river systems. The underlying high hydraulic conductivity aquifer can thus transmit seepage flow and uplift pressures from the river under and up through the low hydraulic conductivity blanket. In the Mississippi River Basin, an extensive study into riverine geology and levee underseepage over two decades began with field investigations, laboratory modeling, and analytical modeling summarized in Technical Manual (TM) 184-1 (USACE 1941) and culminated in TM 3-424 (USACE 1956). Other seminal references from this effort include Bennett (1946), Middlebrooks and Jervis (1947), Barron (1948), TM 3-304 (USACE 1949), and TM 3-430 (USACE 1956). The design approach using blanket theory presented in TM 3-424 attempts to prevent the formation of sand boils by maintaining blanket integrity and has mitigated blanket instability as well as backward erosion piping for many miles of levee subject to underseepage both within and outside of the Mississippi River Basin, when it has been properly evaluated and appropriate mitigation measures were implemented. The general cross-section of typical levees considered in these studies is included here as Figure 6-2.

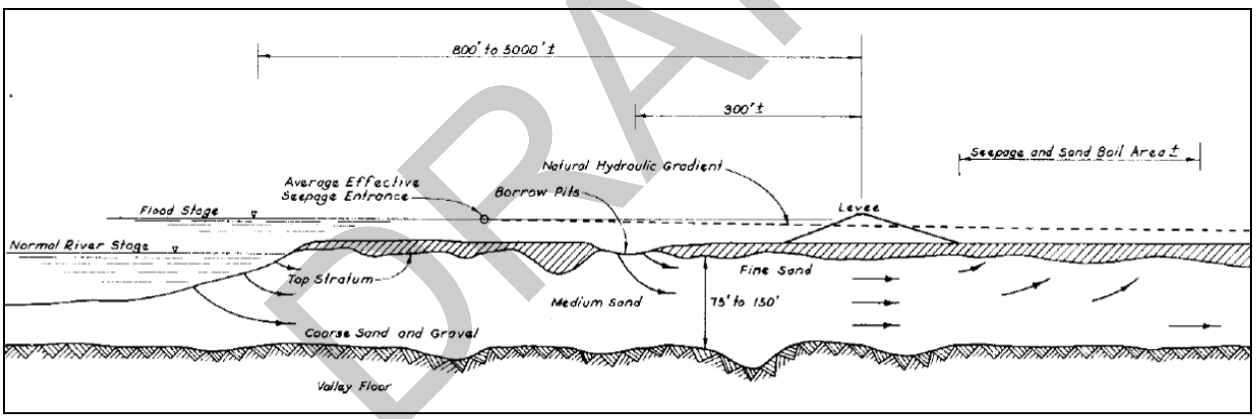

Figure 0-2. Generalized Mississippi River Levee Cross-Section from TM 3-424 (1956).

6.4 Through-Seepage Considerations. Common levee through-seepage conditions that require evaluation include:

a) Through-seepage through levee embankment leading to internal erosion or slope instability.

b) Through-seepage within the bedding and backfill along and/or adjacent to utilities or penetrations such as conduits leading to internal erosion.

c) Through-seepage along the interface with structures such as pump stations, flood walls, roadways, bridges, and control structures leading to internal erosion.

d) Through-seepage around a seepage control measure (such as cutoff wall and seepage berms), if not properly designed, leading to internal erosion.

e) Through-seepage due to localized weakness in the levee embankment due to animal burrowing and/or tree roots leading to internal erosion.

f) Through-seepage in cracks formed due to differential settlements or desiccation or shrinkage. Earthquake-induced cracking can also form due to liquefaction, lateral spreading, and differential settlement at geologic contact or different fill zones. These cracks can lead to the initiation of internal erosion.

6.4.1 Through-seepage has generally been controlled through the proper selection, placement and compaction of levee materials. When a phreatic surface daylights on the landside slope, through-seepage can cause either internal erosion, surficial erosion, and/or slope stability issues, especially if levee side slopes are relatively steep.

6.5 Methods of Analysis.

6.5.1 Analysis Conditions. Analyses should be conducted to evaluate levee seepage for a wide range of conditions in the field to: 1) assess seepage exit and internal erosion conditions; and 2) export pore-water pressures from a seepage analysis (i.e., finite element method seepage analysis) to a slope stability analysis to evaluate stability as described in Chapter 7. When the results of the analysis are being used to evaluate against deterministic seepage criteria, steady-state (e.g., constant flood loading and seepage conditions reach a steady state) seepage conditions shall be used. Non-steady-state seepage conditions may be used during Phase 2 of the risk-informed approach.

6.5.2 Load Cases. The analyses should be conducted for a range of load cases to evaluate the levee performance for seepage under a variety of flood loading conditions. As described in Chapter 1, at a minimum analysis should be performed for the following waterside flood loading conditions:

a) Design water surface elevation

b) As-Constructed (Top of levee)

6.5.2.1 There may be other water levels of interest that designers should analyze for consideration during the final evaluation and design. This may potentially include different combinations of flood loading at lower levels, consideration of water surface above the top of levee during overtopping, and range of landside tailwater or ponding conditions.

6.5.2.2 The annual chance of exceedance of each water level analyzed during design shall be determined and documented for each analyses cross section. This information will be necessary for completion of the final evaluation and design of the levee.

6.5.3 Finite Element and Finite Difference Methods. Modern personal computers have the capability to complete sophisticated numerical analyses of a wide range of problems founded on complex geological conditions using finite element method (FEM) and finite difference (FD). For simplicity, the term FEM is used throughout the rest of this chapter to include both FD and FEM, because the results from either should be interchangeable and for the evaluation of levee seepage, the use of FEM is more prevalent than the use of FD. The results of FEM seepage analysis include pore pressures that are often incorporated into a levee slope stability analysis as described in Chapter 7.

6.5.3.1 An example FEM analysis to replicate the profile shown in Figure 0-2 is illustrated in the upper portion of Figure 6-3. The general approach for conducting these analyses is described in Appendix D and involves calculating head and associated pressures anywhere in the foundation, such as depicted by the equipotential lines that represent contours of equal total head shown on the lower portion of Figure 6-3. Flow lines became less important when traditional design approaches shifted to FEM analysis, which often does not depict flow lines. Because design analysis often focused solely on seepage pressures and the prevention of sand boils, the lack of flow lines was not considered important. Flow lines are included in Figure 6-3 based on locations selected by the analyst to approximate equal flow volumes in the areas between one flow line and the next.

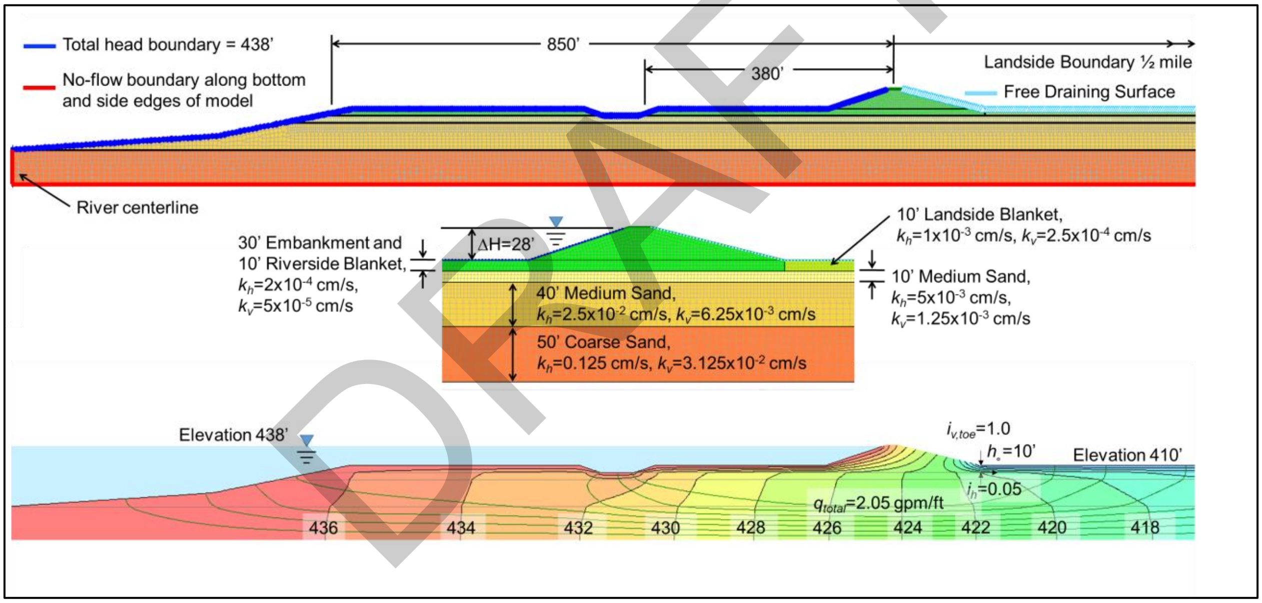

Figure 0-3. Seepage FEM Analysis of a Generalized Levee Cross-Section: a) FEM Model for Seepage Analysis; b) Steady-State Seepage Results with Equipotential Lines and Flow Lines

6.5.4 Blanket Theory Method. Blanket theory (BT) was developed in the 1940s and has been used since the 1950s to evaluate levee underseepage (TM 3-424). Using BT, underseepage controls described in Section III of this chapter have been designed for many thousands of miles of river levees throughout the United States. A description of BT, including a comparison with FEM, is included in Appendix D. Where stratification can be reasonably approximated by two horizontal layers and boundary conditions in a FEM model that matches the assumed conditions for BT, the two approaches yield similar results. However, judgment is required to transform the presence of intermediate silty sand or sandy silt layers into either part of the blanket or part of the aquifer with the 2-layer BT approach that can result in errors (Wolff 2002). FEM can better handle the presence of sloping or intermediate layers, and other site-specific complexities in the foundation, and 3-D FEM analyses can further account for bends in levee alignment and known features of limited extent.

6.5.4.1 BT does not approximate pore pressures in either the blanket or levee soils, and judgment is required when a simple piezometric line is used to determine pore pressures in slope stability analyses. Where BT results are not consistent with pore pressures determined with FEM, FEM results are preferred.

6.5.5 Other Seepage Analysis Methods. Prior to the advent of modern computers and seepage analysis software programs, seepage analyses were performed manually utilizing methods that made many simplifying assumptions and depended on the analyst's professional judgment. Hand-drawn flow nets, electrical models, classic well drawdown equations, method of fragments (used to develop BT), and other approaches have been successfully used to design levee seepage control measures. Most levee seepage analyses performed today use either FEM, BT, or a combination of FEM and BT. A detailed listing of seepage analysis methods is provided in EM 1110-2-1901.

6.5.6 Limitations in Seepage Analyses Methods. Simplified cross-sections developed from limited subsurface information may not capture foundation conditions (e.g., localized geologic variability) or embankment conditions (e.g., defects) in that can significantly alter the levee performance. Appendix D describe levee seepage analyses using either BT or FEM, and list differences between conditions where one or the other of these two approaches should be used, along with simplifying assumptions needed to perform these analyses. Any seepage analysis should be verified by comparison with simple hand calculations, other models, or field performance where available. Although the list is not exhaustive, some limitations to seepage analysis common to many levees are included in the following paragraphs.

6.5.6.1 Three-Dimensional Seepage Conditions. Most seepage analysis are performed using two dimensional (2-D) methods that does not account for variation in topography, geology, sources or sinks along the length of the levee. Three-dimensional (3-D) seepage conditions such as concentration of seepage is likely to occur at the inside bends in levee alignment and near the end of any seepage mitigation feature (i.e., cut-off wall, seepage berm, or line of relief wells). Unfortunately, there is no approach to adjust the results 2-D analyses to 3-D conditions because the relationship between 2-D and 3-D is sensitive to foundation conditions (i.e., depth of the pervious layer, thickness of a less pervious layer, hydraulic conductivity, etc.). Although 3-D FEM analyses can account for the concentration of seepage, the common levee seepage analysis method is to use 2-D analytical tools with observation and judgment. Published recommendations such as a 10 to 30 percent increase in vertical exit gradient on the inside of a bend in levee alignment (URS 2015) or decreased well spacing near the ends of a finite line of relief wells (EM 1110-2-1914) or extending cutoff wall lengths (plan view) to extend seepage path are a few examples of qualitative assessments used to account for 3-D effects.

6.5.6.2 Point Bar Deposits. The geomorphology for point bar deposits along the Mississippi River and other major waterways in the United States is particularly difficult to model accurately, where the blanket is heterogeneous and composed of clay filled swales with sandy ridges between the swales. Regardless of orientation, seepage concentrates along the edge of swales as shown in Figure 6-4 from TM 3-424. While sand boils have long been understood to be more likely to form in these locations, it is important to understand that the 3-D concentration of seepage, once a defect has formed, will also make progression to breach more likely. In instances where the long dimension of the swales is parallel to the levee or intersects it at a small angle, seepage is particularly concentrated in the sandy ridges where the edges of the swales intersect the levee toe as shown in Figure 0-5 from TM 3-424. Clay plugs and channel fillings, also depicted Figure 0-5, concentrate seepage with similar effect but considerably accentuated, owing to their greater thickness and width. Modeling and evaluation of point bar deposits can be complicated and requires input from an experienced designer, but this evaluation may be useful in cases where likelihood of poor performance due to seepage concentrations is high.

6.5.7 Additional Considerations.

6.5.7.1 Observational Method and Levee Seepage Performance. As discussed in Chapter 1, for geotechnical failure modes, for both design and reliability assessments, the epistemic uncertainty (“unknown-unknown”) challenge is common and well-known to the profession and has been addressed through a classic inductive-reasoning approach referred to as the “Observational Method” (Peck 1969). Depending on the depositional environment and details of the design, it is recognized that an unidentified minor geologic detail has some likelihood of existing, but it cannot be economically identified at the time of design and/or possibly during construction, and explicitly accounted for in the analysis model. If observed performance deviates substantially and is worse than predicted, it is likely that the model used to predict performance needs to be updated. Additional investigations may be necessary to 1) refine the model and 2) re-estimate future performance and reliability and, if necessary, design additional measures to improve reliability. To confirm that performance is consistent with design expectations, anticipated piezometer measurements, flow measurements, and associated high-water inspections to confirm performance should be developed and included in the O&M manual for the subject levee. Levees that have performed satisfactorily during significant flood events often have a greater reliability at that level of loading than untested levees. However, levees and levee foundations that are poorly maintained and significantly impacted by damaging vegetation, animal burrows, and progressive deterioration due to multiple flood events that trigger internal erosion, may be less reliable in future flood events.

6.5.7.2 Piezometers. Where feasible, piezometers should be installed along selected portions of the levee system to monitor and confirm performance during flood events. During high-water events, a riser pipe to enable measurement of the free-standing height of water may be needed when pore pressure is higher than the top of the piezometer. Automated instrumentation of piezometers allows for remote, real-time measurement of pore water pressure, which then may enable rapid evaluation of field conditions and response to potential problems. Automated instruments may also overcome problems accessing piezometers during high water events. A set of piezometers across the footprint of the levee and up to 300 feet landward of the levee toe, especially in places with thin blankets, midway between relief wells, and in the vicinity of other seepage control features provide valuable data used to calibrate seepage models and assess levee performance related to underseepage during high water events. The use and function of piezometers are described in Chapter 1 of this manual and in EM 1110-2-1908. Although there is no minimum requirement for inclusion of piezometers, these instruments have the potential to both reduce performance uncertainty and improve risk-informed decisions related to flood operations.

6.5.7.3 Seepage Flow Rates. Measurements of seepage flow rates, often recorded in units of volume per unit time per length of levee, may also be used to help verify design assumptions. Where drainage features are employed, observed seepage should be clear; the presence of sediment indicates possible internal erosion of embankment or foundation soils. Where flow is either turbid or much larger than predicted, the condition should be closely monitored and documented to evaluate changes in condition and assist in decisions about employing risk reduction measures, particularly during flood events.

6.5.7.4 High-water Inspections. Comprehensive visual observations during high-water events are essential to detecting ongoing internal erosion and seepage-related problems before they progress to levee breach. While piezometers and seepage control features tend to be focused in limited areas, comprehensive inspections should cover the entire length of the levee, including adjacent landside areas. Where consequences are high, people should walk along the landside toe looking for sand boils, soft areas, sloughing, and areas of general seepage. Where flow is transporting sediments and/or is much larger than predicted, the condition should be closely monitored and documented to evaluate changes in condition and assist in the decision to initiate flood fighting activities and/or employ emergency risk reduction measures and/or notify emergency responders of potential need to begin evacuation processes. For more information on flood fighting and performance monitoring and documentation, see Appendix J.

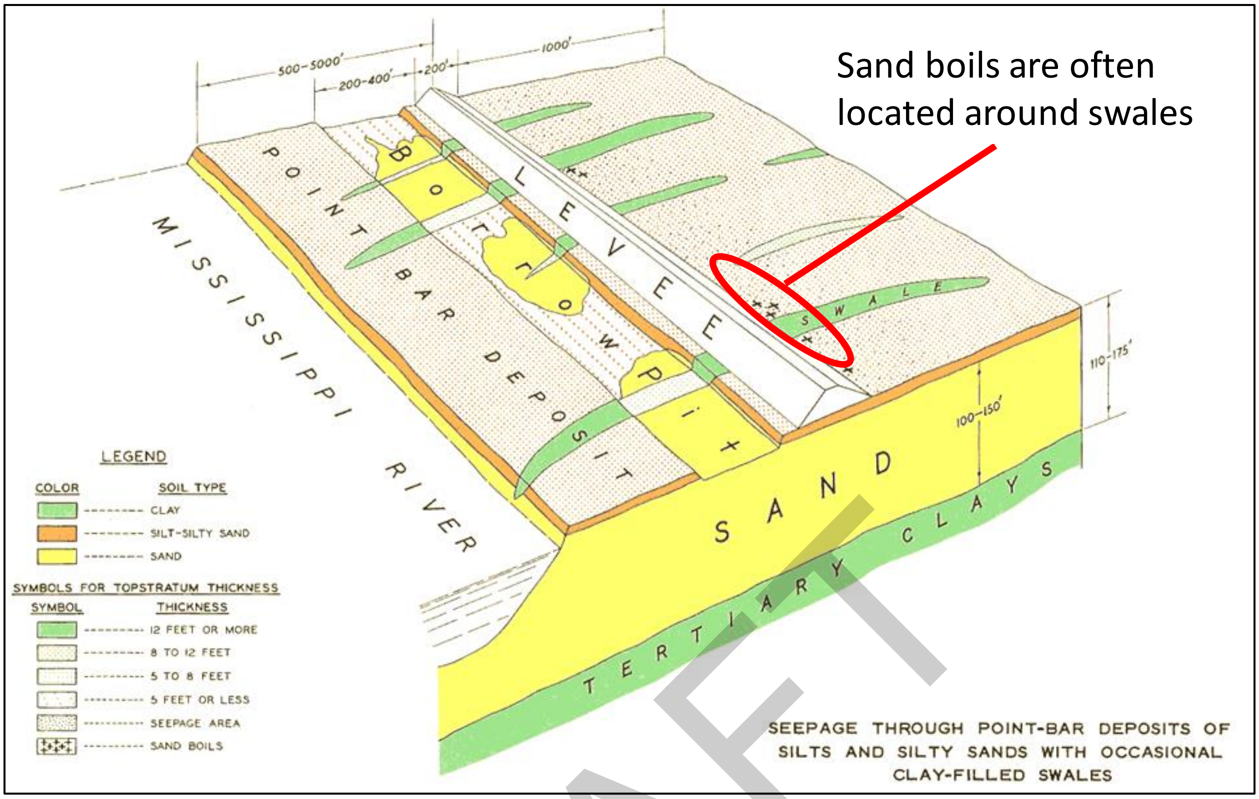

Figure 0-4. Ridge and swale topography in point bar deposits concentrates seepage and results in more observed sand boils (TM 3-424).

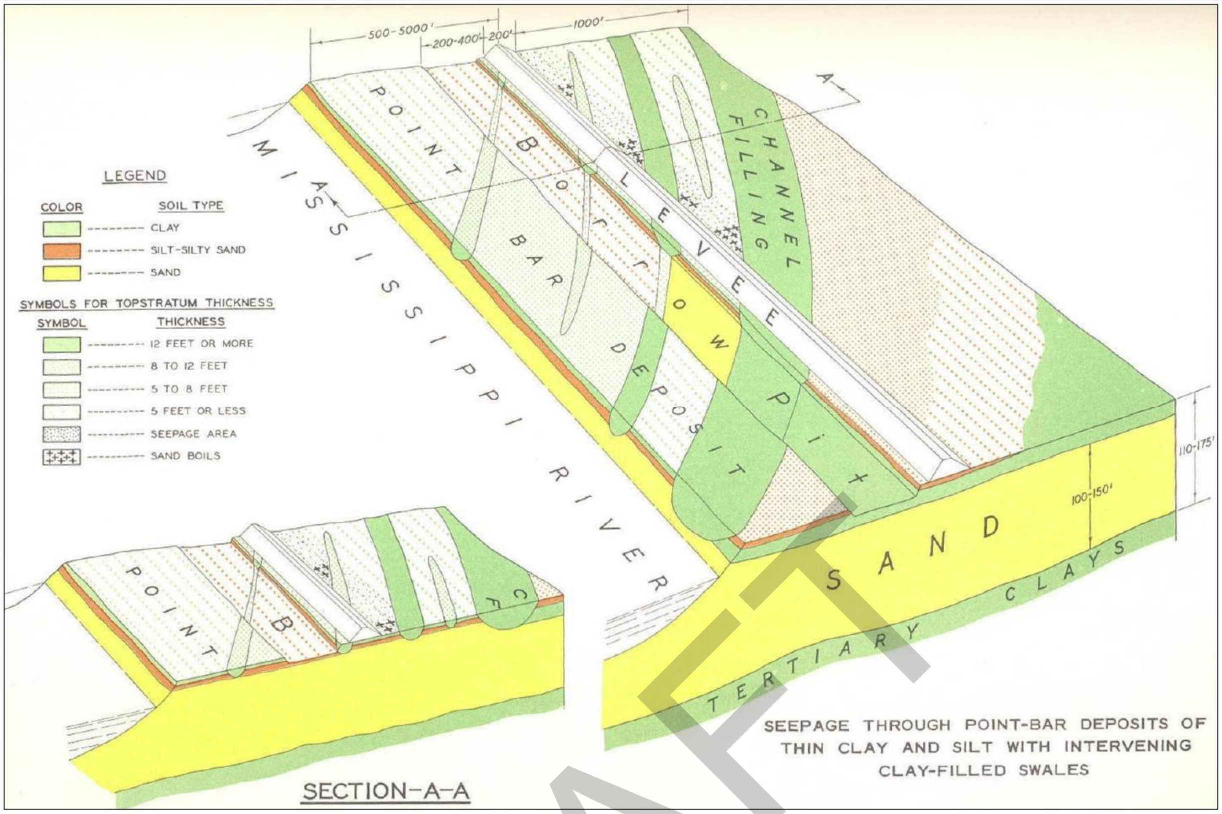

Figure 0-5. Orientation of clay filled channels and swales further accentuate seepage problems (TM 3-424).

6.6 Deterministic Seepage Criteria.

6.6.1 General. Deterministic seepage criteria provided here addresses the initiation and progression of internal erosion failure modes described in Section II. These criteria are only applicable to underseepage conditions where there is a foundation layer susceptible to backwards erosion piping. These criteria are only utilized while performing the initial (i.e., Phase 1) evaluation and deterministic design for levee seepage. If the levee foundation consists only of fine-grained, low hydraulic conductivity soils (e.g., CL or CH), the thickness of the blanket is greater than 2 to 3 times the height of the levee or does not include a layer of soil susceptible to backwards erosion piping, initial evaluation and deterministic design for levee seepage is not required. The final evaluation and design should be done using a risk-informed approach described in Section II.

6.6.2 Limitation. The basis for deterministic underseepage criteria has long been the critical vertical hydraulic gradient factor of safety method as described by Terzaghi and Peck (1948), defined in Section 6.6.3. Deterministic criteria are founded in the concept that backwards erosion piping initiates due to the formation of an unfiltered seepage exit, from a computational standpoint, when the gravitational resistance of the blanket is exceeded by seepage forces. The presence of a confining blanket results in excess head in the aquifer landward of the levee, and the excess head dissipated with seepage across the blanket results in gradient-based seepage forces. TM 184-1 identified the principal factor for sand boil formation as a pervious substratum with sufficient pressure to result in a hydraulic gradient through the blanket greater than the critical vertical gradient. TM 3-424 compares the observation of heavy seepage and sand boils with gradient across the blanket calculated from measured pressure in the substratum. These observations illustrate that geotechnical site investigations will not always discover defects or weaknesses that can cause poor seepage-related performance. Defects or weaknesses represent points of minimum resistance to seepage or preexisting unfiltered exits through the blanket which may include:

a) Animal burrows and crawfish holes,

b) Relic sand boils,

c) Blast holes from seismic surveys,

d) Fence posts and other excavations,

e) Rotten stumps and/or roots, and/or

f) Desiccation or shrinkage cracks (if not closed before high water events).

6.6.3 Initiation Criteria. The intent of the initiation criteria is to prevent underseepage conditions causing an unfiltered seepage exit occurring landward of the levee (e.g., at the landside levee toe) such that progression of backward erosion piping occurs leading to breach of the levee. The initiation criteria are based on factors of safety using the vertical exit gradient as shown in Equation 6-1 below. Parameters z_t and h_x used in the equation are also shown Figure 0-6.

$$FS_{vg} = \frac{i_{cv}}{i_v} = \frac{\gamma' z_t}{\gamma_w h_x} \tag{6-1}$$

where:

FS_{vg} = factor of safety based on vertical gradient
i_{cv} = critical vertical gradient = γ'/γ_w
i_v = vertical exit gradient at point of interest= h_x/z_t, typically the landside toe
z_t = vertical distance to surface, typically the landside blanket thickness
h_x = excess head (above hydrostatic) at the point of interest, typically bottom of blanket (h_o at the embankment toe and h_x at a distance x from the embankment toe)
γ' = average effective (or buoyant) unit weight of blanket (overlying soil) = $\gamma_{sat} - \gamma_w$
γ_{sat} = saturated unit weight of blanket limited to no more than 112.5 lb/ft^3
γ_w = unit weight of water

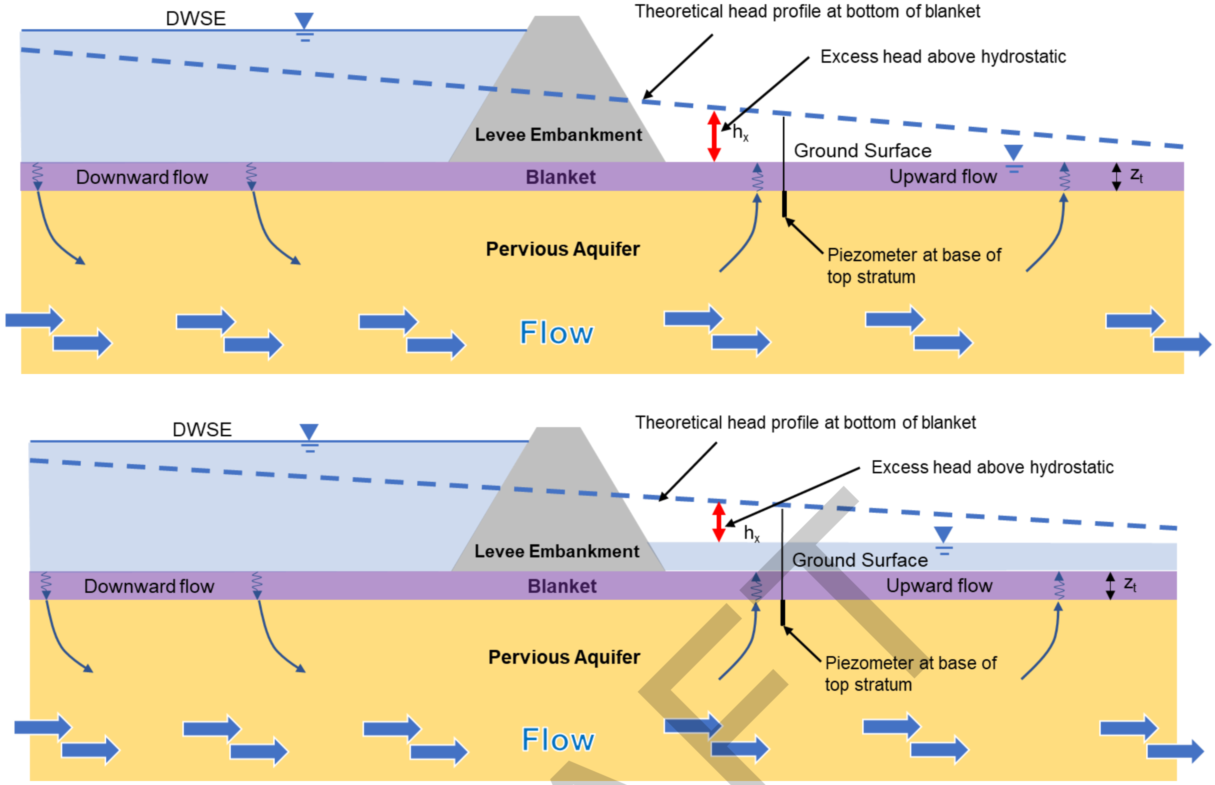

Figure 0-6. Illustration of a generalized levee section with confining blanket overlying a pervious aquifer. The top diagram shows tailwater at the landside ground surface for measuring excess head. The bottom diagram shows tailwater above the landside ground surface.

6.6.3.1 The initiation criteria at the landside levee toe, shown as criteria 1 in Figure 0-7, is a FS_{vg} of 1.6 for the DWSE load case. Note that the designation of the "levee toe" location can be subject to different interpretations, especially when non-uniform levee surface and toe area topography exists. Engineering judgment should be used in the designation of the "levee toe" as well as appropriate FS_{vg} at that location. For example, if a ditch exists near the landside toe and reduces the blanket thickness, FS_{vg} should be evaluated at this location. If there is no blanket or the blanket has been compromised, the progression criterion in Section 6.6.4 should be used to guide the initial deterministic evaluation and design.

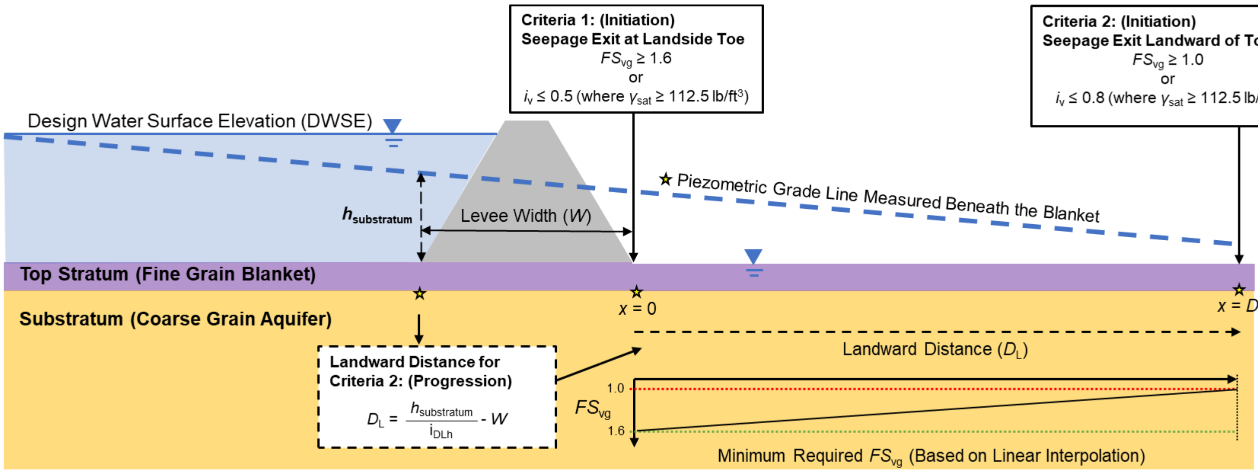

Figure 0-7. Illustration of Deterministic Seepage Criteria.

6.6.3.2 The initiation criteria at a landward distance of D_L from the landside levee toe, shown as criteria 2 in Figure 0-7, is a FS_{vg} of 1.0 for the DWSE load case. D_L is associated with the progression criteria which is defined in the subsequent section. The initiation criteria between the landside levee toe and a distance of D_L is linearly interpolated from 1.6 to 1.0 respectively. This applies to seepage berm lengths, ditch or canal locations, areas where the blanket thins, or topographic depressions less than a distance D_L from the landside levee toe.

6.6.3.3 Lower factors of safety (i.e., less than 1.6) are allowed landward of levee toe, with the thought that intervention is likely to prevent an unfiltered seepage exit that initiates landward of the landside toe from progressing to catastrophic levee breach. Initiation of sand boils (e.g., unfiltered seepage exit) will likely occur at certain critical flood stages when vertical gradient factors of safety are near or less than unity. The distance where low FS_{vg} occurs should be far enough away from the levee so that intervention can be successful, and progression of backward erosion piping is unlikely to occur.

6.6.3.4 In situations where the saturated blanket unit weights are 112.5 pcf or greater, a saturated unit weight of 112.5 pcf should only be used in Equation 6-1. In these situations, vertical exit gradient criteria of 0.5 and 0.8 can be used in lieu of factors of safety for the landside levee toe and landward distance of D_L respectively. It is recommended the blanket saturated unit weight and thickness be verified through subsurface investigations and laboratory testing as these have significant impacts on the deterministic evaluation and design.

6.6.3.5 $FS_{vg} < 1.0$ at locations greater than the distance from the levee toe, D_L, may be present for some projects. This may be the result of landside ditches, canals, swales, lower elevation areas, or localized condition where the blanket is effectively thinner. The presence of lower factors of safety further away from the levee toe should be verified and considered during final evaluation and design. In such cases where lower factors of safety exist at a distance beyond D_L, the O&M manual should incorporate the assumptions on intervention and describe appropriate actions.

6.6.3.6 Generally, FS_{vg} at the bottom of ditches, canals, or other low areas is evaluated with the assumption of no landside ponding. If landside ponding in ditches, canals, or other low areas is assumed for the initial deterministic evaluation and design, a full discussion justifying why/how the ponding elevation will be maintained should be provided. The project's O&M manual should contain directions on maintaining the required ditch water level as well as observation requirements.

6.6.4 Progression Criterion. The intent of the progression criterion is to prevent progression of backward erosion piping (BEP) in the event an unfiltered seepage exit occurs landward of the levee (e.g., at the landside levee toe) such that it leads to breach of the levee. Horizontal hydraulic gradient, i_h, as defined in Equation 6-2 is an indicator of where BEP is likely to progress to breach. Parameters $h_{substratum}$, D_x, and W used in Equation 6-2 are also shown in Figure 0-7.

$$i_h = \frac{h_{substratum}}{D_x + W} \tag{6-2}$$

where:

$h_{substratum}$ = total head at the base of the top stratum or blanket at the waterside toe of the levee based on a seepage analysis.
D_x = the distance from the landside levee toe to the unfiltered seepage exit.
W = levee width

6.6.4.1 The progression criterion is met in the initial deterministic evaluation and design when an unfiltered seepage exit is not expected to occur within a landward distance of D_L from the landside levee toe. An unfiltered seepage exit is likely to occur (based on empirical observations documented in TM 3-424) when the FS_{vg} reaches 1.0. In order to satisfy the progression criterion in the initial deterministic evaluation and design, FS_{vg} should be ≥ 1.0 at a distance (D_L) from the landside levee toe using equation 6-3.

$$D_L = \frac{h_{substratum}}{i_{DLh}} - W \tag{6-3}$$

where:

D_L = the distance from the landside levee toe.
$h_{substratum}$ = total head at the base of the top stratum or blanket at the waterside toe of the levee based on a seepage analysis.
i_{DLh} = horizontal hydraulic gradient to select D_L as discussed below.
W = levee width

6.6.4.2 An unfiltered seepage exit may also occur due to lack of a blanket or daylighting of a pervious substratum (for example, in a landward drainage ditch). In these conditions, backward erosion piping can initiate at very low horizontal gradients, resulting in a progressive erosion of subsurface materials that eventually develops into an eroding pipe that may eventually cause collapse and breach of the levee.

6.6.4.3 Equation 6-3 requires selection of a design horizontal hydraulic gradient, i_{DLh}, based on material properties of the substratum (e.g., BEP layer). Table 0-1 provides recommended design horizontal hydraulic gradients for coarse-grained soils (i.e., SP, SP-SM, etc) based on the coefficient of uniformity (C_u) and fines content (e.g., percent finer than 0.075 mm). The selection of i_{DLh}, should be based on results from subsurface investigations and laboratory tests. The substratum beneath the levee can be highly variable (i.e., fine and coarse layers, layers with either low or high fines content, etc) and the material properties of the layer that results in the lowest i_{DLh}, should be used for the progression criterion.

6.6.4.4 It is recommended that adequate subsurface investigation and laboratory testing be conducted within the substratum to identify and characterize all coarse-grained foundation soil layers susceptible to BEP. If little or no data is available for the characterization of the substratum, i_{DLh} of 0.02 is recommended. If data is available to adequately characterize the substratum, i_{DLh} may be selected using Table 6-1. Background on the values presented in Table 0-1 as well as methods to determine the likelihood of progression of BEP is provided following subsections.

Table 0-1. Design Horizontal Gradient

Characterization of the Substratum	i_{DLh}
$C_u \leq 2$ and fines content ≤ 5%	0.02
$C_u \leq 4$ and 5% < fines content ≤ 10%	0.05
All other coarse-grained soils	0.1

6.6.4.5 For foundations capable of producing a large quantity of flow during flood events, designers and analysts are cautioned that 3-D concentration through a confining layer at a sand boil may be capable of removing foundation material even where average horizontal gradient is low or creep ratio is high. For example, Figure 0-8 shows a sand boil that could not be contained at the toe of the landside berm on Kaskaskia Island Levee on the Mississippi River immediately prior to breach in 1993, even though the average horizontal gradient in the underlying backward eroding material was very low and the creep ratio was fairly high.

Figure 0-8. Attempts to sandbag a sand boil at the Kaskaskia Island Levee. This containment attempt was not successful, and the levee breached minutes later (Photo courtesy of USACE St. Louis District).

6.6.4.6 Background on Progression Criterion. Empirical methods are available to evaluate the likelihood of backward erosion piping progressing to breach. The practitioner should be thoroughly familiar with Article 58 "Mechanisms of Subsurface Erosion" of Terzaghi et al. (1996), and in particular, Section 58.5 "Means for Avoiding Subsurface Erosion." Another valuable source for information on the no-blanket condition is Duncan et al. (2011b). However, as noted below, the empirical evaluation methods have limitations when applied to field conditions and should not be relied upon in design. Engineering judgement should be applied to situations where unconfined seepage is exiting cohesionless slopes or foundations, and a method to reduce the potential for backward erosion piping such as a filter/drain blanket or shallow cutoff should be incorporated into the design.

6.6.4.6.1 Bligh's Creep Ratio and Lane's Weighted Creep Ratio are empirical methods to assess the likelihood of BEP based on observations of seepage performance for a range of soil types. An evaluation of creep ratio is informative where the levee consists of fine-grained material and no landside blanket exists. The method is not appropriate for a compromised confining layer overlying a confined aquifer. The creep ratios are described in EM 1110-2-1901. TM 3-424 recommends adding seepage control measures where no landside blanket exists, creep ratios are lower than threshold values, and flow is greater than 2 gallons per minute per foot (gpm/ft) of levee. The purpose of creep ratio is to assess conditions where it might be prudent to lengthen the flow path "so that the velocity of the seepage water as it emerges on the downstream side is insufficient to remove foundation material" (Lane 1935). Duncan et al. (2011b) state that while informative, creep ratio is considered a "quick-and-dirty" check rather than a rational method of analysis. In addition, threshold values of creep ratio do not include a factor of safety. The state-of-practice is to use rational methods, based on blanket theory, flow nets, or FEM analysis, and the greatest remaining value of creep ratios lies in indicating the relative erosion potential of various soil types.

6.6.4.6.2 More recently, methods based on horizontal gradient have gained more attention for evaluating potential for backward erosion of cohesionless soils, specifically based on the research of Schmertmann (2000) and Sellmeijer (1988, 2006, 2011). The research performed by both Schmertmann and Sellmeijer involved sand flumes in the lab to study the gradient across the structure required to achieve a complete breach. Schmertmann's work has been extended (Allen 2018) and analyzed for use in risk assessments (Robbins and Sharp 2016 and O'Leary and Robbins 2020). Full scale field tests (van Beek et al. 2010) confirmed the retrograde erosion postulated by Terzaghi and later studied by Schmertmann and Sellmeijer. The renewed interest in these methods is helping to bring forward these experimental observations and incorporate considerations that were previously acknowledged but not widely applied, such as gradational coefficients of uniformity.

6.6.4.6.3 Schmertmann provided "lab-to-field" correction factors in a simplified approach using an average or global gradient that is commonly used with risk assessments, but also proposed a more rigorous design method based on site-specific flow nets to investigate local gradients near the advancing pipe head. The Sellmeijer method employs a 'best fit' equation for a two-dimensional seepage regime applicable to situations that have uniform boundary conditions parallel to the embankment centerline (an exposed ditch or no confining layer). The research by Sellmeijer is still ongoing and it may be improved to better account for three dimensional (3-D) concentrations of flow and some of the other deficiencies with either creep ratio or average gradient approaches. It should be noted that these methods for estimating critical horizontal gradient assume 2-D exits such as in the side of a ditch or the toe or the toe of an embankment without a confining layer (planar exit). For 3-D exits such as boils, the 2-D critical horizontal gradient must be reduced by a factor of 2 (Van Beek et al. 2015).

6.6.5 Considerations for Coastal Hazards. The duration, antecedent conditions prior to flooding, and geologic conditions in coastal levees are different than in most riverine levees. Due to the short duration associated with hurricane events, the inability to work in hurricane winds, and the general inaccessibility of much of the levee system during a hurricane, there will be no opportunity to conduct levee patrols or to flood fight levee or floodwall distress to prevent failure (HSDRRS 2007). Thus, more stringent levee seepage deterministic criteria may be needed for coastal levee seepage. Appendix D includes discussions on seepage analyses for coastal levees.

Section II
Levee Seepage Final Evaluation and Design

6.7 General. As discussed in Chapter 1, there are two steps required for the design of levees. The first step is for a deterministic evaluation and design to be completed outlined in Section I of this chapter. The deterministic evaluation and design are only the starting point for the levee seepage evaluation and design. Experience has demonstrated that strictly complying with deterministic levee seepage criteria does not always produce levees that have the expected level of reliability. It has also been observed that blind compliance with deterministic criteria sometimes results in levees that are "overdesigned" with features that are not actually improving levee performance and reducing levee risk. The second required step in design is to evaluate and adjust the initial design using a risk assessment. The risk assessment will also serve the basis for deciding to upscale the design to be more robust than required by the deterministic criteria or to be downscaled to allow for use of reduced design criteria. A formal design deviation must be submitted in compliance with applicable USACE policy for approval before deterministic criteria and factors of safety lower than outlined in Section I will be considered acceptable.

6.7.1 The purpose of the final evaluation and design is to ensure the goals (i.e., flood risk reduction, costs, environmental benefits, etc) of the levee project are achieved. During the final evaluation and design, the levee project will be assessed for internal potential failure modes and the risk (e.g., hazard, performance, consequences) associated these failure modes are estimated. For levee seepage, identifying and evaluating internal potential failure modes is required as part of final evaluation and design. Guidance on evaluating internal erosion potential failure modes is provided within the subsequent paragraphs in this section. It is important that internal erosion potential failure modes for a levee project are evaluated for a range of flood loading conditions including the loading cases listed in Section 6.5.2. The duration of the flood event should also be considered when evaluating performing the final evaluation and design. Refer to Chapter 1 on the process that should be followed for final evaluation and design.

6.8 <u>Internal Erosion Potential Failure Modes Overview.</u>

6.8.1 Potential failure modes due to internal erosion are caused by water flowing through a soil mass leading to erosion within the soil, which ultimately leads to increased flow velocities and volumes through preferred seepage paths, increased potential for further soil erosion and loss of levee integrity. The potential for internal erosion is present in most levee systems and may be the most significant issue affecting levee performance. Internal erosion can involve the levee embankment, the levee foundation, or both.

6.8.2 Best Practices in Dam and Levee Safety Risk Analysis (USACE et al 2019 or latest version) describes the failure mechanisms for internal erosion which is summarized in the following paragraphs.

- Backward Erosion Piping (BEP). Occurs when soil erosion (particle detachment) begins at a seepage exit point and erodes backwards (upstream), supporting a "roof" along the way. As the erosion continues, the seepage path gets shorter, and flow concentrates, leading to higher gradients, more flow, and the potential for erosion continues to increase. Four conditions must exist for BEP to occur: 1) flow path or source of water; 2) unprotected or unfiltered exit; 3) erodible materials within the flow path; and 4) continuous stable roof forms allowing pipe to form. BEP is the most critical form of internal erosion mechanism in levees, as it involves progression of a subsurface pipe towards the water body.

a) Concentrated Leak Erosion. Involves erosion of the walls of an opening (crack) through which concentrated leakage occurs.

b) Soil Contact Erosion. Involves the selective erosion of fine particles from the contact with a coarser layer caused by the passing of flow through the coarser layer parallel to the contact.

c) Internal Instability – Suffusion and Suffosion. Both suffusion and suffosion are internal erosion mechanisms that occur with internally unstable soils. Suffusion involves selective erosion of finer particles from the matrix of coarser particles (that are in point-to-point contact) in such a manner that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles. With suffusion, there is typically little or no volume change. Suffosion is a similar process but results in volume change (voids leading to sinkholes) because the coarser particles are not in point-to-point contact. This condition may require consideration of BEP, cracking and concentrated leak erosion, or contact erosion.

d) Internal Migration (Stoping). Occurs when the soil is not capable of sustaining a roof or pipe. Soil particles migrate downward primarily due to gravity, but may be aggravated by seepage or precipitation, and a temporary void grows in the vicinity of the initiation location until a roof can no longer be supported, at which time the void collapses. This mechanism may be repeated progressively until the levee is breached or the downstream slope is oversteepened to the point of instability. Since by definition roof support is lacking, this mechanism typically leads to a void that may stope to the surface as a sinkhole.

6.8.3 Table 0-2. General Internal Erosion Potential Failure Modes List for Levees includes a general list of internal erosion potential failure modes and. Additional information for the other potential failure modes can be found in the Best Practices in Dam and Levee Safety Risk Analysis.

Table 0-2. General Internal Erosion Potential Failure Modes List for Levees

- Foundation underseepage leading to heave internal erosion of foundation materials
- Levee embankment through-seepage leading to internal erosion or instability of levee embankment materials
- Seepage and internal erosion around conduits (pipes) or penetrations through the levee and underlying foundation
- Seepage and internal erosion into conduits or pipes

*Note: List above is not exhaustive or may be not applicable to every levee.

6.9 Internal Erosion Potential Failure Mode Evaluation.

6.9.1 Backward erosion piping is generally the most common internal erosion failure mode in many geomorphic conditions. The erosion process begins at the seepage exit, possibly near the landside levee toe, in landside ditches or borrow pits near the levee. A defect in a confining layer forms as a result of heaving/blowout, desiccation cracking, animal burrows, excavation, or other penetrations through a blanket layer. Backward erosion piping is often manifested by the presence of sand and sand boils, especially if the seepage discharge surface is nearly horizontal. On sloping surfaces, the slow downward movement of soil particles is a sign of the development of this critical condition. A typical event tree of internal erosion potential failure mode by backward erosion piping through a levee foundation is shown in Figure 6-9. The event tree can be tailored to site-specific conditions.

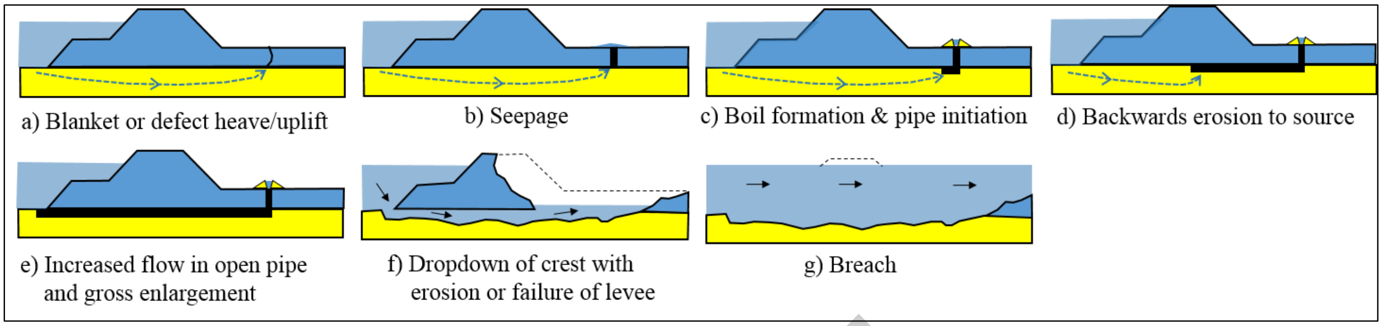

Figure 0-9. Internal Erosion of the Levee Foundation Materials due to Levee Underseepage (Adapted from van Beek et al. (2010)).

6.9.2 For vertical flow situations, Terzaghi et al. (1996) showed that heave will occur when a zero effective stress condition occurs in soils subject to upward seepage. After heaving, an unfiltered exit that allows the backward erosion piping process to begin may be established. The most common basis for levee design with pervious foundations is to prevent this vertical heave condition. In some regions, it has been implicitly assumed that if backward erosion initiates to form a sand boil (that is, particle detachment occurs), then it will progress to form a pipe, especially under repeated loading from successive flood events. The seepage control measures are designed often focused on stopping the potential for heave; levee failure was considered unlikely if heave was prevented. Risk assessment for levee designs explicitly evaluates all the ensuing steps after heave/uplift have occurred, including presence of erodible materials, materials capable of sustaining a roof, and the presence of adequate horizontal gradients to sustain the erosion process. If backward erosion piping progresses from the seepage exit to the flood source, it may lead to collapse of the levee into the pipe and subsequent overtopping breach.

6.9.3 Once an unfiltered exit exists either due to heave or defects, the hydraulic conditions under which backward erosion will progress to form an expanding erosion pipe must also be considered. Test results from studies by Weijers and Sellmeijer (1993), Schmertmann (2000), and Sellmeijer et al. (2011) have shown that backward erosion can initiate and progress at very low horizontal gradients, especially for unfiltered fairly uniform, fine to medium sands. Where seepage may emerge near-horizontally, BEP can progress with average horizontal gradients as low as 0.05. It has been noted that this critical horizontal gradient for fine sand is approximately the same as the inverse of the Bligh's Creep ratio for the same material (that is, $i_{critical\ for\ fine\ sand} \sim 1/\textit{Bligh's Creep Ratio}$ for fine sand) and thus appears to be consistent with empirical field observations of when BEP is more likely to progress to failure (See Section 6.6.4 for more information on creep ratios).

6.9.4 The assessment of the hydraulic condition for progression of backward erosion piping considers point and average/global gradients in the foundation and the critical gradient for particle transport (to advance a pipe to the river) from Sellmeijer et al. (2011) and Schmertmann (2000). In general, the likelihood of backward erosion piping is:

- Decreased by increasing particle size (i.e., sand is more likely than gravel)
- Decreased by increased coefficient of uniformity (i.e., well graded materials are less likely than uniformly graded materials)
- Decreased by increasing relative density
- Decreased by decreasing hydraulic conductivity
- Decreased by angularity of the particles (i.e., rounded particles are more likely than angular particles)
- Decreased by increasing fines content (i.e., clean sands are more likely than sands with fines).
- Increased by the increased thickness of the piping layer
- Increased by presence of an underlying layer of higher hydraulic conductivity
- Increased by increased horizontal to vertical hydraulic conductivity ratio (i.e., anisotropy)
- Increased by the increase in potential defects in the confining layer
- Probably not significantly affected by confining stress in the erodible layer

6.9.5 Culverts or other pipe penetrations through the levee or levee foundation pose additional threat; they may allow for internal erosion potential failure modes such as concentrated leak erosion and internal migration in the surrounding pipe bedding and backfill. These pipe penetrations can lead to the presence of high hydraulic conductivity zones or cracks in the surrounding soil ("flaws") as it is often difficult to compact materials around penetrations. Seepage paths can occur in the surrounding soil leading to internal erosion. The seepage can exit at the end of the culvert or through a defect in the culvert such as open pipe joint or cracks in the pipe. For many existing levees, these pipe penetrations may be improperly installed, severely corroded, or damaged, which increases the likelihood of internal erosion. Additional information about culverts and other pipe penetrations through levees is provided in EM 1110-2-2902.

6.9.6 The deterministic evaluation on FS_{vg} is often also used to inform the likelihood of sand boils forming through a confining blanket. Sand boil formation is typically one node of many evaluated in an event tree format (see Chapter 1) to estimate the risk of levee breach due to BEP, resulting inundation, and consequences. The most current state-of-the practice methods for evaluating backward erosion piping (BEP) should be used. The risk assessment should include an evaluation of all factors that will affect progression of BEP including the thickness of the substratum layer, whether there are any concentrations of seepage in the foundation or at the exit, and the overall geomorphology of the levee site.

Section III
Levee Underseepage Control

6.10 General. Underseepage in pervious foundations beneath levees may result in (a) excessive pore water pressures beneath a landside blanket, (b) sand boils, (c) backward erosion piping beneath the levee itself, and (d) levee slope instability. Underseepage problems are most acute where a pervious substratum underlies a levee and extends both landward and riverward of the levee and where a relatively thin blanket exists on the landside of the levee. In addition, many levees have an adjacent, waterside borrow channel that completely penetrates the surficial waterside blanket, is incised into the aquifer, and provides a hydraulically efficient seepage entrance point for the pervious substratum. Common seepage control measures for levee underseepage are:

a) cutoff trenches or walls,

b) waterside impervious blankets,

c) landside seepage berms,

d) trench drains, and

e) pressure relief wells.

6.10.1 These methods will be discussed generally in the following paragraphs. Methods used to evaluate seepage berms and trench drains are presented in Appendices E and F. Additional information on seepage control in foundations including cutoffs, impervious blankets, seepage berms, relief wells, and trench drains are given in EM 1110-2-1901 and EM 1110-2-1914. The seepage control measure selected for a levee will be site specific and include such considerations as space available to construct the seepage control measure, a sponsor’s ability to maintain the selected seepage control measure, and access to construct and maintain the selected seepage control measure. It is important to understand the potential negative effects climate and seismic conditions can have on seepage control features, some of which are listed in Sections 11.13 and 11.14.

6.11 Cutoff Trenches or Walls.

6.11.1 A seepage cutoff beneath a levee to restrict or reduce seepage through pervious foundation strata is one of the most effective, but also generally the most expensive means of mitigating seepage problems. Positive cutoffs (e.g., a seepage cutoff feature that fully penetrates into an underlying aquiclude layer) may consist of shallow excavated trenches backfilled with compacted earth or slurry trenches and are usually located between the waterside toe and the levee centerline. Experience has shown that seepage remediation measures other than positive cutoffs generally become more economical when the pervious stratum reaches a depth greater than 40 feet below the foundation grade, except in areas of high real estate acquisition costs, where the small footprint of a slurry trench may provide economic advantages. However, economics are not the only factor to consider and there are instances where conventional excavator-constructed cutoffs of about 90 feet deep have been used for levees. Other construction methods, such as Deep Mixing Methods (DMM), have been used to construct cutoffs to depths of 150 feet. The economics of cutoff walls are constantly changing as the technology changes; what was too expensive on an earlier project may become viable on a later one.

6.11.2 Additional information on cutoff design is available in EM 1110-2-1901 and other available best practices documents for seepage control cutoffs for dams and levees. Sheet-piling is not usually watertight due to leakage at the interlocks but can significantly reduce the possibility of piping of sand strata in the foundation. If sheet-pile cannot be installed by conventional driving operations, and methods such as jetting or use of a mandrel are required, care should be taken to reliably fill any potential voids between the pile and in-situ soil. Any void along the side of a cut-off will be a pathway for BEP to develop, particularly where the cutoff does not extend into a less pervious layer. Open trench excavations can be readily made above the water table, but if they must be made below the water table, dewatering systems will likely be necessary. Cutoffs made by the slurry trench method can be made without a dewatering system, and the cost of this type of cutoff should be favorable in many cases in comparison with costs of compacted earth cutoffs.

6.11.3 Designers and constructors should note that a slurry trench that extends through an embankment has the potential to cause severely damaging hydraulic fracturing either through weak layers in the embankment or the foundation. Special measures to avoid hydraulic fracturing may be necessary, such as increased viscosity of support slurry and/or pre-construction levee degrading. ER 1110-1-1807 provides guidance on drilling through levees to prevent hydraulic fracturing, erosion, filter/drain contamination, heave, or other mechanisms that must be avoided during slurry trench construction. Degrading the levee to facilitate trench construction may require the use of a coffer dam where potential consequences in the leveed area and frequency of overtopping are both high.

6.11.4 The selection of a type of cutoff wall system should consider site-specific conditions. These factors may include but should not be limited to factors such as 1) location of cutoff wall (centerline or waterside toe), 2) thickness of the aquifer layer, 2) platform width required for construction, 3) level of levee degrade necessary to provide access to construction equipment and prevent hydrofracture, 4) thickness and depth of wall, 5) stabilizing agent (such as cement) considering cutoff wall methods and geologic conditions, 6) impact of horizontal or vertical mixing on the resulting product for in-situ mixing methods, 7) stability of open trench for open excavation methods, 8) long term performance of cutoff wall considering environmental conditions (such as water-chemical interaction, cracking, and erosion potential), and 9) performance in seismic events. Considering these factors should help the designer to select an appropriate cutoff wall system to achieve design objectives.

6.11.5 Stability for cutoff wall construction is critical to ensure the integrity of the levee is not compromised during construction. Important factors that must be considered include in-situ soil weights and strengths, the hydrostatic distribution of slurry pressure, ground water elevation, filter cake formation, duration of excavation, construction surcharge, and soil arching. Panel lengths are crucial for stability because soil arching will occur at the ends of the panel, providing higher lateral resistance to soil collapse along the length of the slurry-filled trench. There is no arching component to continuous wall construction resulting in a lower factor of safety than an otherwise similar wall constructed in panels. Both drained and undrained conditions must be evaluated to ensure they meet all required factors of safety according to Chapter 7.

6.11.6 Deep cutoffs will interfere with the normal exchange of groundwater between an aquifer and a river that could result in an increase of groundwater levels landside of the levee during non-flood periods. Conversely, a deep cutoff could decrease groundwater levels and restrict recharging landside of the levee during periods of flooding. The effect of the cutoff on groundwater levels and resultant impacts such as the flooding of below-grade structures and reduction of water supply should be considered. Additionally, as less water would seep out of the river channel with a deep cutoff, potential hydraulic impacts such as increased downstream flow/stage should also be considered, though it is expected they will often be minor.

6.12 Waterside Blankets. Riverine levees are frequently situated on relatively fine-grained impervious to semipervious soils overlying pervious sands and gravels. These surface strata constitute impervious or semipervious blankets when considered in relation to seepage control. If these blankets are continuous and extend riverward for a considerable distance, they increase the effective entrance distance and can effectively reduce seepage flow and seepage pressures landside of the levee. Where underseepage is a problem, waterside borrow and channel dredging operations should be limited in depth and proximity to levee to prevent penetrating the impervious blanket in the levee vicinity. If there are limited areas where the blanket becomes thin or pinches out entirely, the blanket can be made effective by placing impervious materials in these areas. The effectiveness of the blanket depends on its thickness, length, distance to the levee waterside toe, and hydraulic conductivity. Blanket effectiveness can be evaluated by seepage analysis methods. Borrow material may be more efficiently used on the landside as a seepage berm, and low-lying areas on either side of levees are often designated wetlands that require mitigation when filled. Placing fill on the waterside in many areas is not feasible due to environmental and habitat concerns. Protection of the waterside blanket against surface erosion may also be important, and methods to evaluate erosion and erosion mitigation are discussed in Chapter 9.

6.13 Landside Seepage Berms.

6.13.1 General. Seepage berms differ from stability berms by their intended function. They look similar in the field (although seepage berms are typically much wider than stability berms), and both types often provide both stabilizing and seepage-related benefits, but they are constructed for different purposes. Seepage berms are primarily designed to counter underseepage and high uplift pressures in the levee foundation, whereas stability berms are meant to provide predominantly counterbalancing weight to prevent slope instability and address through-seepage issues. Discussion in this section will focus on seepage berms only.

6.13.1.1 If seepage uplift pressures in pervious deposits underlying the blanket landward of a levee become greater than the effective weight of the blanket, sand boils are likely to occur. Where space is available, the construction of landside berms can mitigate this performance problem by providing (a) the additional weight needed to counteract these upward seepage forces, (b) the additional seepage-path length such that uplift pressure at the toe of the berm is below tolerable levels, and (c) additional seepage-path length, reducing the likelihood of BEP progression. A free-draining berm designed to be filter-compatible with the existing blanket, as described in Section 6.13.2.4, addresses initiation of BEP through the blanket within the footprint of the berm. Pervious or semipervious seepage berms allow for upward seepage through the blanket that relieves pressure in the aquifer over the length of the berm. Seepage berms may reinforce an existing blanket or, if none exists, may be placed directly on pervious deposits. A seepage berm also affords some protection against sloughing of the landside levee slope. Berms are relatively simple to construct and require very little maintenance. Seepage berms typically require additional fill material and real estate purchase.

6.13.1.2 Subsurface and topographical profiles must be carefully studied when selecting berm widths. For example, where a levee is founded on a thin blanket and thicker clay deposits lie a short distance landward, as shown in Figure 0-10. 10, the berm should extend far enough landward to overlap the thick clay deposit, regardless of the computed required length. Otherwise, a concentration of seepage and high exit gradients may occur between the berm toe and the landward edge of the thick clay deposit. Similarly, a concentration of seepage would occur where the ground surface rises beyond the seepage berm termination, and the seepage berm should be extended so that it intersects the rising topography. The analyst should be aware of the influence of non-horizontal layering, or geologic and manmade features that inhibit flow and pressure relief through a semi-pervious confining layer and evaluate potential weak areas after berm placement.

6.13.2 Types of Seepage Berms. Four types of seepage berms have been used, with selection based on available fill materials, space available landside of the levee proper, performance requirements, and relative costs.

6.13.2.1 Impervious berms. A berm constructed of impervious soils restricts upward seepage flow through a semipervious blanket and the corresponding pressure relief. Consequently, impervious berms need to be wider and thicker than other types of berms due to increased uplift pressures beneath the blanket. However, the berm can be constructed to the width, thickness and weight to provide the required effective stress necessary to have an adequate factor of safety against uplift. While a large amount of material is needed to construct an impervious berm compared to other types of berms, an impervious berm may be an economically feasible option due to local borrow source availability.

6.13.2.2 Semipervious berms. A berm constructed of material that has an in-place vertical hydraulic conductivity equal to or greater than that of the blanket is considered semipervious. Based on the varied nature of blanket soils, it is recommended that berms designed with semipervious berm equations be constructed of sandy materials no finer than silty sands unless there is high confidence in blanket soils over the entire berm footprint. The intent of this type of berm is to increase effective resisting stresses, and some seepage will pass through the berm and emerge on its surface. A semipervious berm results in less pore pressure increase than an impervious berm. However, since the presence of this berm creates additional resistance to flow, it results in a larger increase in subsurface pressures at the levee toe than a pervious sand berm.

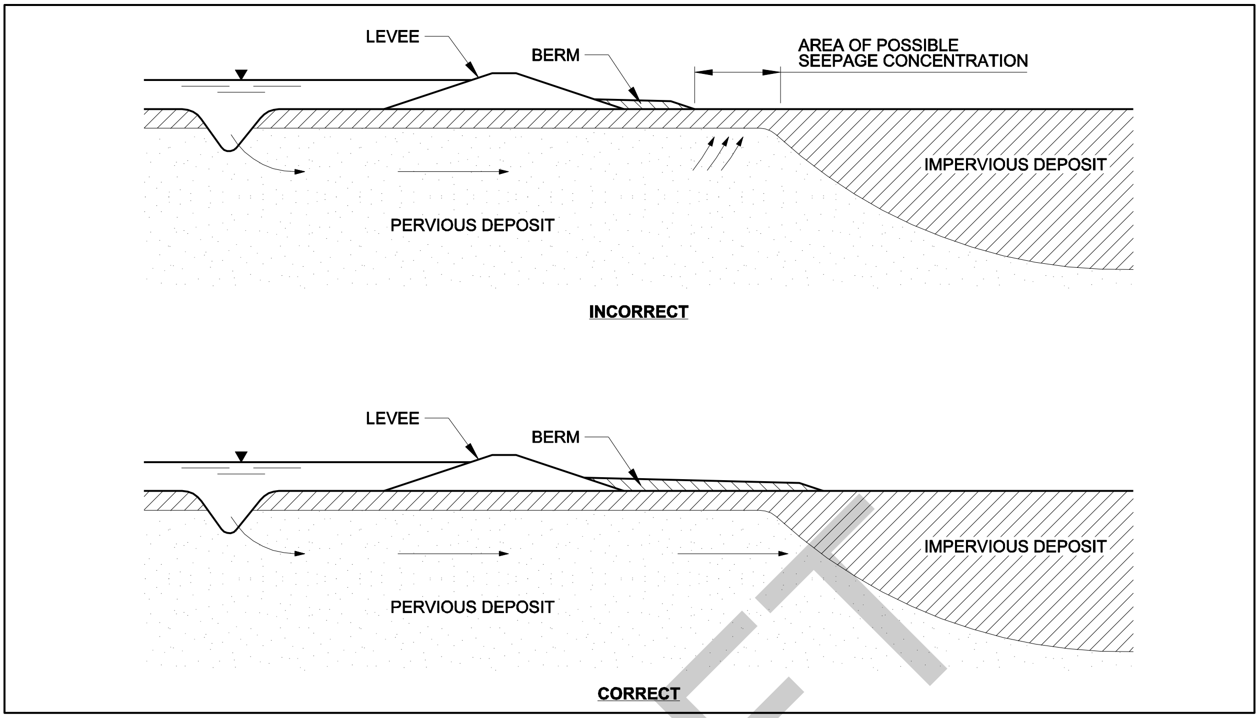

Figure 0-10. Example of Incorrect and Correct Berm Width According to Existing Foundation Conditions (thicker impervious deposit at a distance).

6.13.2.3 Sand berms. Sand berms generally have higher hydraulic conductivity contrasts with the blanket layer than a semipervious berm and often require less material and occupy less space than similarly designed impervious or semipervious berms, often providing the same degree of protection. Filter compatibility between the blanket and the sand berm should be considered in the design. While a sand berm will offer less resistance to flow than a semipervious berm, it may also cause an increase in substratum pressures at the levee toe if it does not have the capacity to rapidly conduct seepage flow landward away from the berm in the absence of significant head losses (i.e., if gravity drainage is insufficient, internal ponding can occur). Sand berms are susceptible to both surface and internal erosion if erosion-preventing measures are not included in the design. Sand berms should not be over compacted which could increase seepage resistance and substratum pressures. Additionally, confirmation of sand berm gradation should be performed after placement and compaction to verify no breakdown of particles has occurred.

6.13.2.4 Free-draining berms. To be designed as free-draining, berm fill must be placed on horizontal sand (filter) and gravel drainage layers (generally with a terminal perforated collector pipe system), designed by the same methods used for drainage layers in dams. A free-draining berm is essentially a weighted filter which can reduce initiation of BEP for erodible blanket layers. Although a free-draining berm can afford protection against underseepage pressures with less length and thickness than the other types of seepage berms, the cost is generally much greater than the other berm types due to more stringent fill requirements. Nevertheless, the benefits of having a well-designed and constructed filtered exit for seepage may significantly reduce the likelihood of developing internal erosion and improve expected performance.

6.13.3 Seepage Berm Design. Seepage berm design equations, criteria, and examples are presented in Appendix E.

6.14 Trench Drains.

6.14.1 General. Where a levee is situated on thin deposits of less pervious material underlain by pervious material, a partially penetrating trench drain as shown in Figure 0-11 can improve seepage conditions at or near the levee toe. This type of drainage feature has also been referred to as "pervious trench drain" when intended to intercept underseepage. Where the pervious stratum is thick, seepage will bypass the toe. The analyst will need to evaluate the performance of the structure in light of this seepage. Where the thick pervious stratum has a good hydraulic connection with the source, even a small portion of aquifer flow will overwhelm an inadequately sized drain. Consequently, the main use of a trench drain is to control shallow underseepage and protect the area in the vicinity of the levee toe. Pervious trench drains may be used in conjunction with relief well systems; the wells collect the deeper seepage, and the trench collects the shallow seepage. FEMA (2011) is a comprehensive report with a good summary of the current state-of-the-practice for drainage filters and provides valuable case histories where improperly designed or constructed drains have failed. Appendix F covers the analysis and design of trench drains and supplements the following sections.

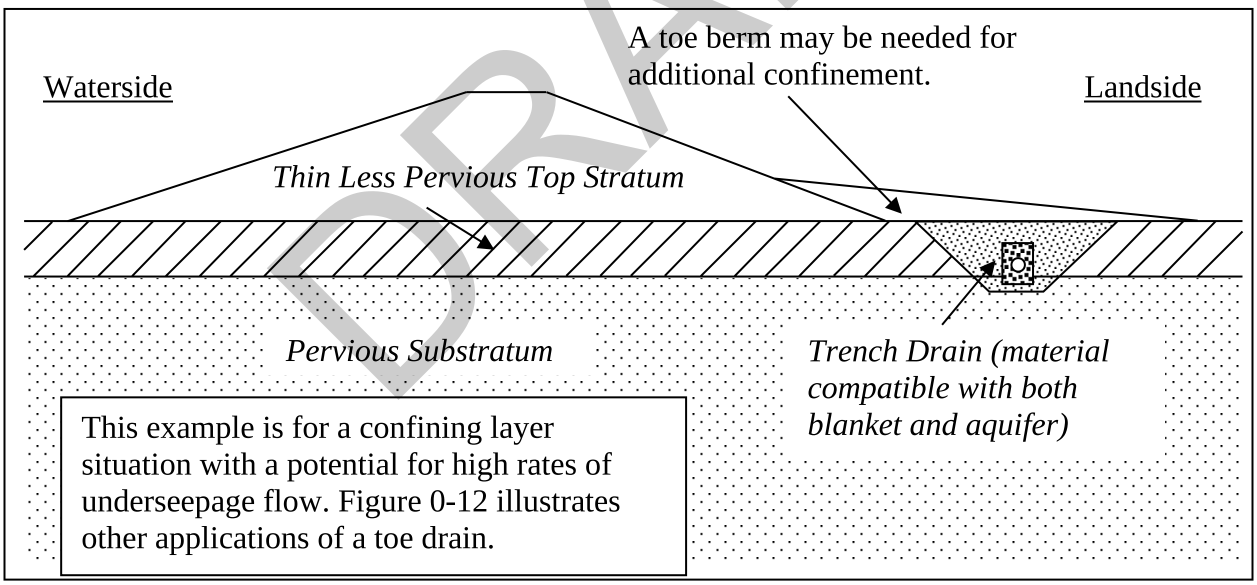

Figure 0-11. Typical Trench Drain at Levee Toe.

6.14.2 Location. Trench drains are generally located at the levee toe as shown in Figure 0-11, but also have been constructed beneath the landside levee slope as shown in Figure 0-12.

6.14.3 Trench drains at the levee toe generally have partial penetration through the aquifer and could have limited capacity. Filter materials for these trench drains should be compatible with both aquifer and blanket. It is convenient to have discharge at the ground surface; however, collector pipes are also used (as shown in Figure 0-11). Cedergren (1989) recommends that the phreatic surface remains within the drain, and thus both the thickness and hydraulic conductivity of the drain must be adequate to convey the resulting seepage flow. A toe berm may be needed for additional confinement. However, a collector system buried beneath a berm may be more difficult to repair or replace if it becomes clogged.

6.14.4 Trench drains on landward slopes usually consist of a drain at the landward quarter point of the levee, and discharge is provided through a horizontal pervious drainage layer. Unless it is deep enough, the drain may allow excessive seepage pressures to cause uplift at the toe. There is some advantage to a location under the levee if the trench serves also as a construction inspection trench and because the horizontal pervious drainage layer can help to control embankment seepage. The weight of the levee also helps provide confinement stress to ensure filter materials do not float out of the drain. However, a drain buried beneath the levee will be more difficult to repair or replace if it becomes clogged. Horizontal drainage layers also help prevent high pore pressures from developing in the landside embankment slope and may greatly improve stability during long duration floods. As long as the drain exit is properly filtered and maintained this could be a key feature to greatly improve slope stability reliability of a design at potentially a low cost if proper materials are available. Conversely, if the exit becomes clogged, a horizontal drainage layer in the configuration shown in Figure 0-12 could transmit seepage pressure from the substratum to the toe and exacerbate slope stability problems. Trench drain materials need to be filter-compatible with both aquifer and blanket materials, which may require a double-layer filter system. It is also important to know the waterside embankment and shallow foundation conditions to ensure that the drainage layer does not expedite seepage entry or increase seepage quantity during a high-water event if measures are not taken to account for these conditions.

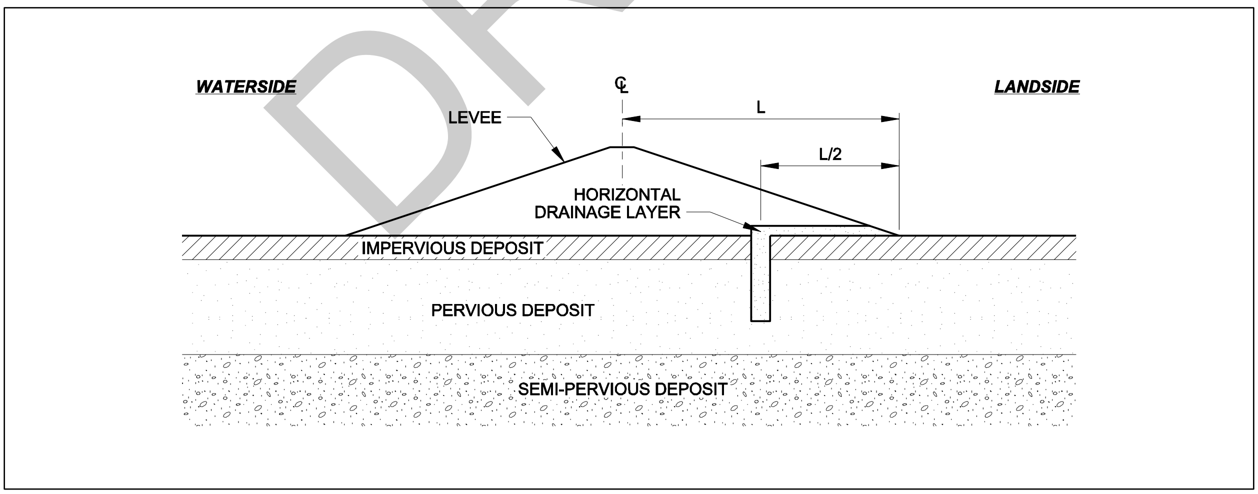

Figure 0-12. Trench Drain Located Beneath Landward Slope.

6.14.5 Geometry. Drain geometry will depend on the volume of expected underseepage, desired reduction in uplift pressure, construction practicalities, geology, and the stability of the material in which it is being excavated. Trench drains have either rectangular or trapezoidal cross-sections. Rectangular drains with vertical side walls are typically used where seepage is expected to be small. Placement of multi-stage filters that are rectangular and vertical may be difficult. Drain widths as small as 2 feet have been used, and trench excavation can be expedited with a trenching machine. However, narrow trench widths will require special compaction equipment and may not allow for adequate inspection of in-situ soils, the importance of which is described in the following sections. The sloping side walls for a trapezoidal drain result in a more compliant contact between drain materials and in-situ soil than the vertical side walls of a rectangular drain. If a problem were to develop, the distance along the contact to the ground surface is larger and flow is spread over a larger area which reduces velocity at the surface. Trapezoidal trench sections are recommended where larger amounts of seepage may occur, a thick confining layer is present, or potential consequences of trench failure are high.

6.14.6 Backfill. The sand backfill for trenches must be designed as a filter material in accordance with criteria given in EM 1110-2-1901. The terms filters and drains are sometimes used interchangeably. Some definitions classify filters and drains by function, where flow through filters is perpendicular to the interface between the protected soil and filter and do not need to have a particular flow or drainage capacity. Trench drains, toe drains, and horizontal discharge layers all collect seepage and conduct it to a discharge point or area, and therefore required adequate discharge capacity. If a collector pipe is used, the pipe should be surrounded by a minimum 1-foot thickness of drain rock having a gradation designed to provide a stable transition between the filter backfill and the perforations or slots in the pipe. Collector pipes must also include drain cleanouts to make them accessible for inspection, cleaning, and rehabilitation. The drain configuration shown in Figure 0-11includes a coarser drainage material embedded within a finer filter material is termed a two-stage filter and is more robust than a single-stage filter. The number of stages in a multi-stage design is dependent on soil type, expected flow, foundation conditions, risk level, and drain geometry. Trench drain backfill must be placed in such a manner as to minimize segregation. Compaction of the backfill should be limited to prevent breakdown of material or over-compaction resulting in lowered hydraulic conductivity.

6.14.7 Contact Erosion.

6.14.7.1 The Best Practices manual (USBR and USACE 2019) defines contact erosion as the selective erosion of fine particles from the contact with a coarser layer caused by the passing of flow through the coarser layer parallel to the contact. Contact erosion will result if drain material is not filter-compatible with in-situ soil. Additionally, Terzaghi et al. (1996) highlight that this type of erosion can occur with flow through the foundation in sedimentary deposits where layers of inorganic silt are in direct contact with layers of clean coarse sand or gravel. If the sand layer is covered with a filter fine enough to prevent the escape of silt, the flow of water out of the sand layer is obstructed. Such conditions of stratification can make it difficult to use filters to reliably prevent subsurface erosion.

6.14.7.2 Practitioners prefer deeper relief wells to shallow trench drains for the control of levee underseepage in large meandering river valleys because they place solid pipe sections, or blanks, through the blanket and any underlying silt layers thereby reducing the potential for internal erosion of silt into and through the sand filter. For example, in the Lower and Middle Mississippi River Valley, the deeper, older Pleistocene substratum deposits are likely to be internally stable. The shallow, more recent Holocene deposits are likely to be heterogeneous.

Blanket thickness is not continuous, is often composed of thin layers of clays, silts, and fine sands, and there are rather abrupt changes in stratigraphy along the length of levee, especially in point bar environments (ridge and swale topography). Even then, special precautions described in EM 1110-2-1914 often need to be taken to better ensure relief wells are functional and potential problems are proactively identified with pump-testing rather than reactively discovered during high water events. Some of these precautions are mentioned in Section 6.15. The Best Practices manual (USBR and USACE 2019) provides a good summary of guidance on evaluation of internal stability for a given sample based on grain-size distribution. However, these approaches may not adequately address contact erosion that can occur between alternating coarse and fine layers.

6.14.8 Discharge Capacity. The discharge capacity of a trench drain must be considered in design. The landside levee toe is a critical location where an inadequately designed drainage feature could quickly lead to catastrophic breach of the levee. The check for discharge capacity is generally performed by assessing a best estimate for foundation hydraulic conductivity and increasing that value by a factor of 10 to 20. An alternate method of discharge capacity check that achieves the same purpose is presented in EM 1110-2-1901 and Cedergren (1989). Both of these methods are presented in the example analyses in Appendix F. It should be noted that an assumption behind Darcy's law is that flow is laminar, while flow through coarse drainage materials becomes turbulent at relatively low gradients, resulting in lower "apparent" hydraulic conductivity. Cedergren (1989) provides correction factors that should be applied to account for turbulence when flow is into a collector system and there is adequate confinement. These correction factors are not appropriate when flow will discharge freely at the ground surface, and a drain that discharges freely at ground surface will need to be sized to ensure flow is laminar.

6.14.9 Collector Systems.

6.14.9.1 The trench is frequently provided with a perforated pipe to collect the seepage, and the decision to include a collector system should consider several factors. The use of a collector system is dependent on the volume of seepage and, to some degree, the general location of the levee. Collector systems are usually not required for agricultural levees but may be more appropriate in urban areas. As stated in 6.14.2, it is preferable to discharge at an elevation lower than the ground surface, which is only possible with a collector system. Unlike dams, where the ground surface often slopes away from the embankment toe and allows for lower-elevation discharge from collector systems, the ground surface landside of levees is generally the same elevation as the embankment toe, which does not facilitate a lower discharge elevation. An advantage of a collector system is that it provides an opportunity to inspect the drain (such as camera inspection) and can be designed to accommodate pumping during high water events by including sumps that extend beneath the collector pipe at riser locations. Manholes have been used that include weirs to allow for flow measurement and settlement basins for collecting sediment entering the system, giving an indication of the severity of potential internal erosion. A disadvantage of a collector system is that there is a potential for pipe sections to separate, thus forming an unfiltered exit of filter material. Another drawback to a collector system is that if the discharge pipe begins to pump filter material during a high water event, it is difficult to ascertain where along the system a problem has occurred. A method to plug or otherwise stop the pipe discharge flow should be considered so the pipe can be taken out of service and allow linear surface discharge should evidence of unsafe conditions be detected.

6.14.9.2 The collector system should include either check or flap valves to ensure surface water does not backflow into the drain. Flap gates should be inspected as part of a regular inspection schedule to ensure they are operating correctly. A collector system should be used where it is undesirable for surface water to enter and contaminate water in the underlying soils.

6.14.10 Bio-fouling. The trench will be subject to the accumulation of microorganisms referred to as "bio-fouling" and may be caused by algae or bacteria, plant roots, animal activity, and mineralization. In many regions, iron bacteria are prevalent in the aquifer and these bacteria thrive wherever groundwater is exposed to air. This condition occurs near the ground water level, which unfortunately is frequently near the ground surface at the toe of a levee. Bio-fouling has resulted in drainage features needing to be replaced periodically. Where fouling can be expected to occur, the frequency and effectiveness of routine cleaning is important as fouling can become so severe that redevelopment to the original efficiency is prevented and replacement may be needed. Where drain pipes are present and fouling occurs, regularly scheduled cleaning and evaluation as well as periodic replacement should be planned. In other locations where drain pipes are not present, periodic replacement of all fouled filter/drain materials should be expected. Bio-fouling can be reduced by locating drainage materials and the slotted portion of collector pipes such that they will remain below the water table. If a collector pipe is not used or is located above the bottom of the trench, a horizontal layer of bio-fouling would impede discharge and reduce capacity of the trench.

6.14.11 Inspection. During construction, it is important to inspect the side walls and bottom of the trench to verify in-situ soils are similar to those assumed in design, particularly with regard to filter gradation requirements. Absent visual confirmation of location, placement, filter compatibility, internal stability of in-situ soil, and drain discharge capacity, the best of design intentions can be overwhelmed by unforeseen conditions in the field. If a collector system is used, it is recommended to inspect the pipe using a camera early on during placement of backfill materials so that any defects can be discovered and corrected without excavating the full depth of the trench. During high water events, it is important to inspect the trench and monitor performance for flow, turbid seepage, and sediment accumulation. A trench drain provides a means for quantitative measurement of seepage to aid in observation/analysis of seepage-related behavior. A flow measuring device such as a weir or flume and a sediment trap upstream of the measurement device should be included at the discharge end of the drain or other locations along the drain alignment. Flow out of the trench should be clear (i.e., no transported sediments); the presence of sediment would indicate potential loss of embankment or foundation soil. Unlike relief wells, which can readily be pump-tested to verify performance, there are few ways to test a collector system during periods of low water.

6.15 Relief Wells.

6.15.1 General. Pressure relief wells, or relief wells, may be installed along the landside toe of levees to reduce uplift pressure which may otherwise cause sand boils and piping of foundation materials. Wells accomplish this by intercepting and providing properly filtered, controlled outlets for seepage that would otherwise emerge uncontrolled landward of the levee. Relief well systems are used where a landside confining layer is present and underlying pervious strata are too deep or too thick to be penetrated by cutoffs or toe drains or where space for landside berms is limited. Relief wells should adequately penetrate pervious strata and be spaced sufficiently close to intercept enough seepage to reduce uplift pressures to allowable levels beyond and between the wells. The wells must offer little resistance to the discharge of water while at the same time preventing loss of any soil due to proper filter design. They must also resist corrosion and be able to withstand cleaning required to prevent or minimize bacterial clogging. Relief well systems can be expanded if the initial installation does not provide the control needed. Also, the discharge of existing wells can be increased by pumping if the need arises. A relief well system usually requires less space and right-of-way acquisition/access compared with the other seepage control measures such as berms. However, wells require periodic maintenance and frequently suffer loss in efficiency with time, probably due to clogging of well screens by muddy surface waters, bacteria growth, or carbonate incrustation. They increase seepage discharge, and a means for collecting and disposing of the discharge must be provided. EM 1110-2-1914 covers the analysis and design of relief wells and supplements information in the following sections.

6.15.2 Design of Relief Well Systems. The design of a pressure relief well system involves evaluation of well spacing, size, and penetration to design a system that will reduce uplift and increase vertical gradient factors of safety between wells to allowable values. Factors to be considered are (a) depth, stratification, and hydraulic conductivity of foundation soils, (b) distance to the effective source of seepage, (c) characteristics of the landside blanket, (d) degree of pressure relief desired, (e) loss of well efficiency with time, and (f) ability of project sponsor to conduct the required maintenance including affordability of maintenance. Relief wells are generally spaced between 50 and 200 feet apart. Guidance on the method used to assess well spacing, size, and penetration is contained in EM 1110-2-1914. Relief wells should be designed so that the vertical gradient through the confining blanket midway between the wells results in FS_{vg} equal to or greater than 1.6 at the levee toe. Loss of efficiency during intervals between maintenance events should be not only be considered but assumed in the design. Landward of the levee toe, the criteria for factor of safety between wells is established based on distance from the toe as described in Section I. Many combinations of well spacing, size and penetration will produce the desired pressure relief; hence, the final selected spacing and penetration must be based on cost comparisons of alternative combinations. After the general well spacing for a given reach of levee has been assessed, the actual location of each well should be established to ensure that the wells will be located at critical seepage points, will fit natural topographic features, and will provide adequate seepage control at the end-points of adjacent well reaches with different well spacings or with no foundation underseepage control.

6.15.3 Well installation. Proper methods of drilling, backfilling, and developing a relief well must be employed or the well will be of little or no use. These procedures are described in detail in EM 1110-2-1914.

6.15.4 Well monitoring and inspection. ER 1110-2-1942 describes relief well monitoring and inspection requirements. Relief wells are subject to bio-fouling described in Section 6.14.10 and require periodic maintenance to rejuvenate their flow capacity.

6.15.5 Other considerations. Discharge from relief wells is generally allowed either to flow onto the adjacent ground or is collected in a ditch (lined or unlined) or closed pipe. From an inspection viewpoint, it is preferable for relief wells to discharge above the ground surface or in an open ditch in order that their proper functioning during high water events can be confirmed.

However, relief wells with higher-invert discharge elevations are less efficient. If a below-grade collector pipe is used, each relief well should be fitted with a removable cover such that the discharge of each individual well can be monitored. If below-grade collection ditches are used, the gradient at the bottom of the ditch (or uplift pressure if the ditch is lined) should meet allowable factor of safety criteria. If a lined ditch is used, pressure relief valves may be required at the bottom of the ditch to protect the ditch from damage. Relief wells are commonly installed along levee areas with previous poor performance. If sand boils have been present in the past, it is not unusual for those same boils to reform even after the installation of surrounding relief wells. If this condition exists, consideration should be given to excavating the boil areas and reconstructing the blanket layer to promote seepage to be collected/relieved by the relief wells.

Section IV
Seepage Control Through Embankments

6.16 General. If through-seepage in a levee embankment emerges on the landside slope, as shown in Figure 0-13a, it can soften fine-grained fill in the vicinity of the landside toe, cause sloughing of the slope, or even lead to internal erosion of embankment materials. FEM results can be used to estimate the location of a levee's phreatic surface breakout point. If a phreatic surface daylights on the landside slope of a levee under a steady-state-seepage condition, it may indicate potential for through-seepage performance problems.

6.16.1 Erodible levee embankment materials (such as dispersive clays, sandy silts, and silty sands) pose an additional challenge because concentrated seepage through defects in the embankment can quickly progress through the various stages of internal erosion, leading to breach.

6.16.2 Sand levees without a waterside impervious layer reach steady-state conditions rapidly and are prone to through-seepage. Seepage exiting on the landside slope would result in high seepage forces, decreasing the stability of the slope. However, through-seepage has been successfully mitigated with sand levees by using sufficiently wide levee sections with flat slopes. The USACE Rock Island District has successfully controlled through-seepage issues for many miles of sand levees with 1V:5H landside and 1V:4H waterside slopes, although these slopes do require maintenance during high water events. Figure 0-14 shows sloughing due to seepage through sand levees with steep slopes. Schwartz (1976) provides details on the pictured sand embankment that is part of a test section on the Iowa River in 1962 with 1V:4H landside and 1V:3H waterside slopes, shown in the left image in Figure 0-14.

6.16.3 In many cases, high water stages do not act against levees constructed of finer-grained materials long enough for the full phreatic surface to pass through the embankment, but the possibility of a combination of high water and a period of heavy precipitation may result in phreatic surface breakout on the landside slope.

6.16.4 If landside seepage or stability berms are required because of foundation conditions, they may be all that is necessary to prevent seepage emergence on the slope. If no berms are needed, landside slopes are steep, and flood stage durations and other pertinent considerations indicate a potential problem of seepage emergence on the slope, provisions should be incorporated in the levee section. These provisions could include horizontal and/or inclined drainage layers or toe drains to prevent seepage from emerging on the landside slope. These drainage features require select pervious granular material and graded filter layers to ensure continued functioning, and therefore add an appreciable cost to the levee construction unless suitable materials are readily available in the borrow areas with only minimal processing required. Where large quantities of pervious materials are available in the borrow areas, it may be more practicable to design a zoned embankment with a large landside pervious zone. This would provide an efficient means of through-seepage control and good utilization of available materials. Additional information on seepage control in earth embankments including zoned embankments and vertical (or inclined) and horizontal drains is given in EM 1110-2-1901.

6.17 Cutoffs. Section 6.11 describes the use of a seepage cutoff beneath a levee to restrict or reduce seepage through pervious foundation strata. A shallow cutoff wall through the levee and upper portion of the foundation can also be an effective means to mitigate through-seepage.

6.18 Toe Drain. A pervious toe (Figure 0-13b) will provide a ready exit for seepage through the embankment and can lower the phreatic surface sufficiently so that no seepage will emerge on the landside slope. A pervious toe can also be combined with partially penetrating trench drains, which have previously been discussed, as a method for controlling shallow underseepage. Such a configuration is shown in Figure 0-13c.

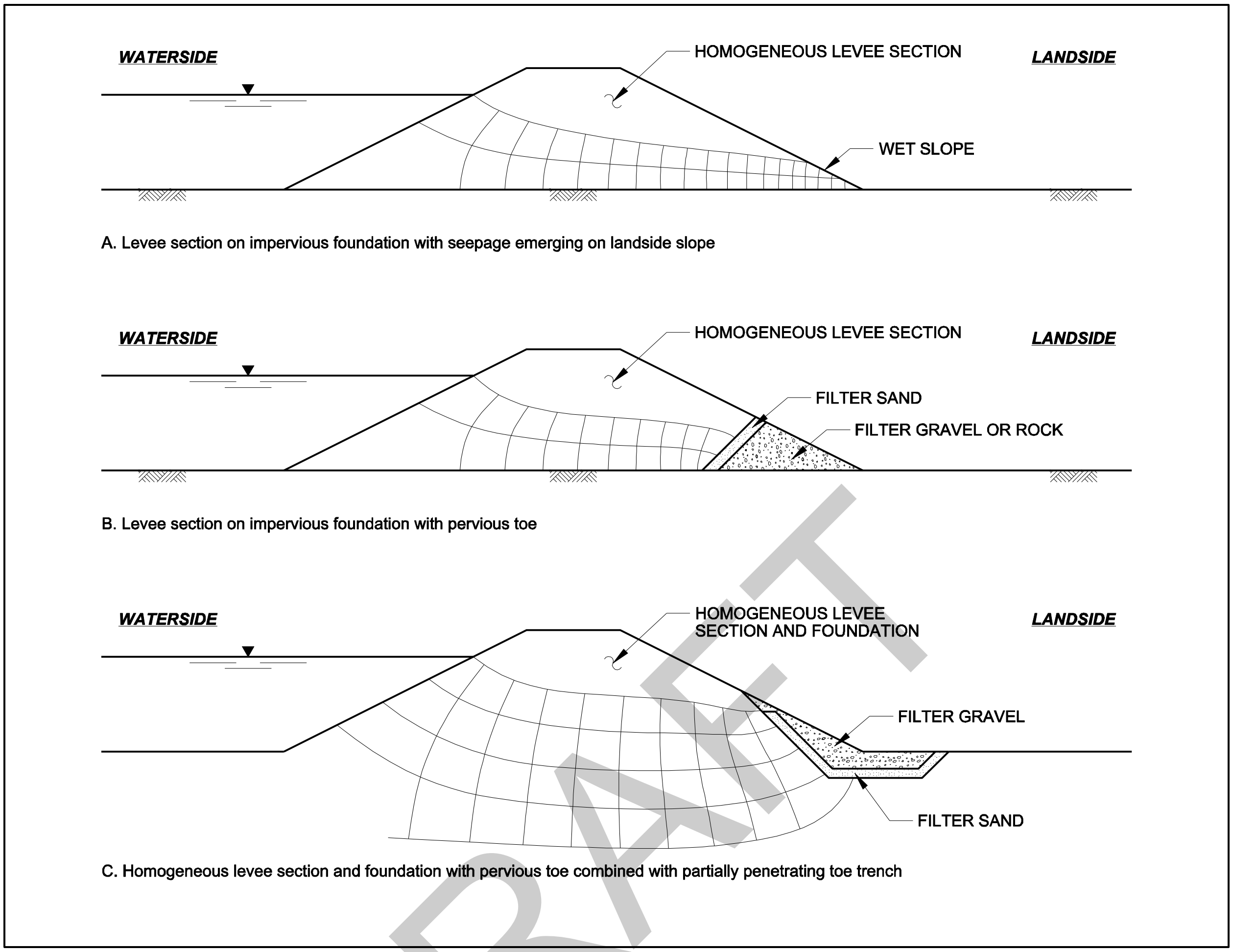

Figure 0-13. Through-Seepage Conditions through Erodible Embankment Materials and Mitigation Measures.

Figure 0-14. Sloughing Due to Seepage through Steep Sand Levee Sections. Photos courtesy of USACE Rock Island District.

6.19 Horizontal Drainage Layers. Horizontal drainage layers, as shown in Figure 0-15a, essentially serve the same purpose as a pervious toe. Horizontal drainage layers are advantageous by comparison because they can extend further under the embankment requiring a relatively small amount of additional material. The amount of additional material required depends on layer thickness and if multiple or double filter layers are needed. They can also serve to protect the base of the embankment against high uplift pressures where shallow foundation underseepage is occurring.

6.19.1 Horizontal drainage layers sometimes also serve to carry off seepage from shallow foundation drainage trenches some distance under the embankment, as shown in Figure 0-12. Horizontal drainage layers also help prevent high pore pressures from developing in the landside embankment slope and may greatly improve stability during long duration floods. As long as the drain exit is properly filtered, has adequate discharge capacity, and is properly maintained, this could be a key feature to improve slope stability reliability of a design, potentially at low cost if proper materials are available. Conversely, a horizontal drainage layer in the configuration shown in Figure 0-12 would transmit seepage pressure from the substratum to the toe and could exacerbate slope stability problems if the exit becomes clogged. If such a measure is selected, it is also important to know the waterside embankment and shallow foundation conditions to ensure that the drainage layer does not expedite seepage entry or increase seepage quantity during a high-water event.

6.20 Inclined Drainage Layers. An inclined drainage layer as shown in Figure 0-15b is one of the more positive means of controlling internal seepage and is used extensively in earthen dams. It is rarely used in levee construction because of the added cost but might be justified for short levee reaches in important locations where landside slopes must be steep, other control measures are not considered adequate, and the levee will have high water against it for prolonged periods. An inclined drain will help reduce through-seepage pore pressures in the landside portion of the embankment. If properly designed as a filter it will also provide protection against internal erosion through a high hydraulic conductivity zone, cracks, or other defects in the embankment. When used between an impervious core and outer pervious shell (Figure 0-15c), it also serves as a filter to prevent migration of impervious fines into the outer shell. If the difference in gradation between the impervious and pervious material is great, the drain may have to be designed as a two-stage filter (See EM 1110-2-1901). Inclined drains must be tied into horizontal drainage layers to provide an exit for the collected seepage as shown in Figure 0-15b and Figure 0-15c.

6.21 Design of Drainage Layers. The design of pervious toe drains and horizontal and inclined drainage layers must ensure that such drains have adequate thickness and hydraulic conductivity to transmit seepage without any appreciable head loss while at the same time preventing migration of finer soil particles. The design of drainage layers must satisfy the criteria outlined in EM 1110-2-1901 for filter design. Horizontal drainage layers should have a minimum thickness of 18 inches for construction purposes.

6.22 Compaction of Drainage Layers. Placement and compaction of drainage layers must ensure that adequate density is attained but should not allow segregation and contamination to occur. Vibratory rollers are generally the best type of equipment for compaction of cohesionless material although tracked equipment and rubber-tired rollers have also been used successfully. Saturation or flooding of the material as the roller passes over it will aid in the compaction process and in some cases has been the only way specified densities could be attained. Care must always be taken to not over-compact to prevent breakdown of materials or lowering of expected hydraulic conductivities. Loading, dumping, and spreading operations should be observed by qualified personnel to ensure that segregation does not occur. Gradation tests should be performed both before and after compaction to ensure that the material meets specifications, and the fines content is not too high. The specifications should indicate that final approval will be based solely on the in-place gradation.

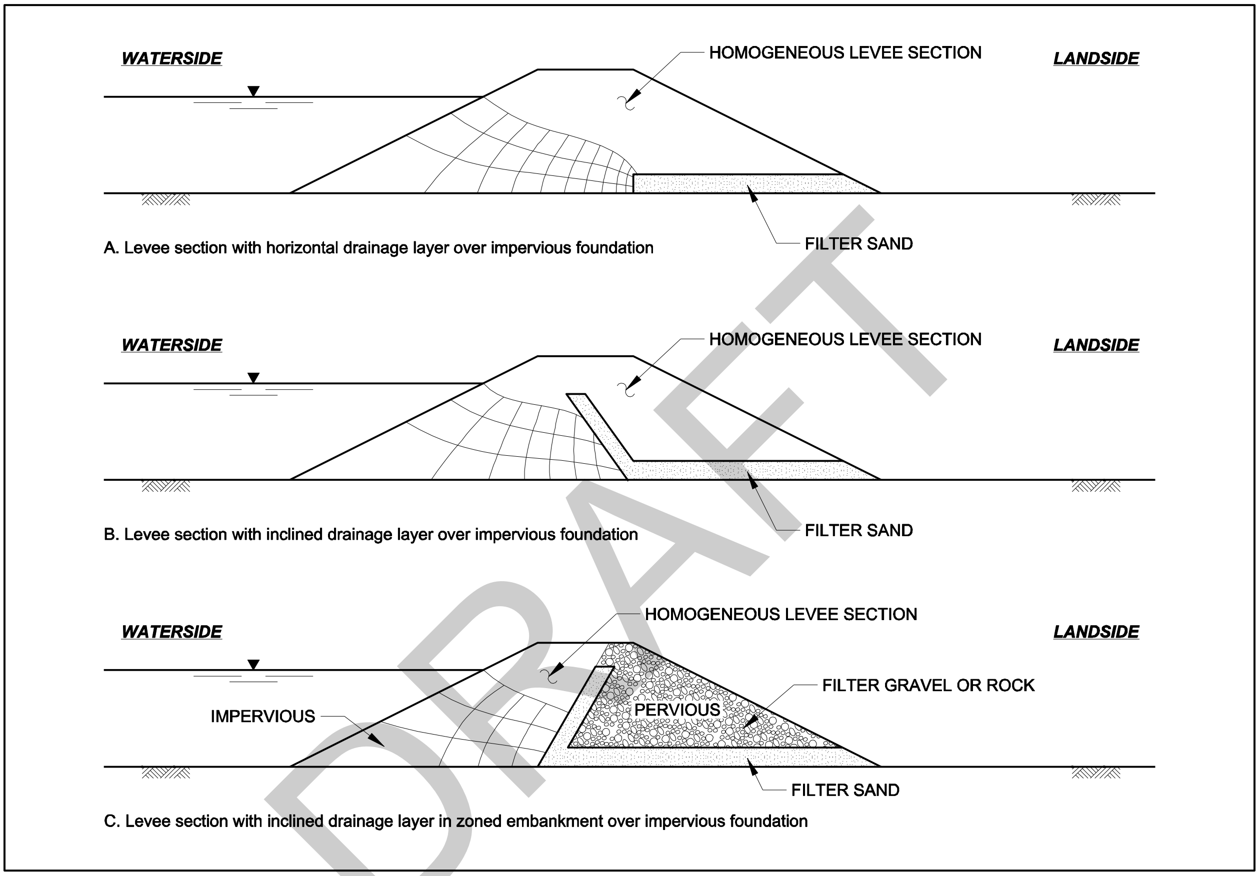

Figure 0-15. Use of Horizontal and Inclined Drainage Layers to Control Seepage Through an Embankment.

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CHAPTER 7

Slope Design

7.1 General

7.1.1 This chapter presents methods to perform stability analyses related to slope design for levee embankments. Analysis conditions and related selection of shear strength, pore water pressure conditions, and deterministic evaluation and design criteria are outlined. More detailed information on applicable shear strengths, methods of analysis, and assumptions are presented in EM 1110-2-1902. The undrained response during flood loading is incorporated in this manual, which is not required in EM 1110-2-1902. This chapter also covers final slope evaluation and design including stability mitigation measures.

7.2 Embankment Geometry.

7.2.1 Minimum Levee Section and Standard Levee Sections.

7.2.1.1 The minimum levee section shall have a crown width of at least 10 feet and side slopes flatter than or equal to 1V:2H, regardless of the levee height. The required minimum levee section dimensions are to provide access for flood-fighting, maintenance, inspection, and for general safety conditions. However, many levees will require wider crown widths and flatter side slopes depending on the results of stability and seepage analyses, embankment materials, vegetative cover, historical performance observations of similar levees in the area, to provide improved vehicular access during flood events, and to meet other project specific operations and maintenance requirements. The factors that should be considered when determining the required levee section for a particular levee are discussed in Sections 7.2.2 and 7.2.3.

7.2.1.2 Many USACE districts have established standard levee sections for particular levee systems, which have proven satisfactory over the years for the local stream mechanics, foundation conditions, and local construction practices. For a given levee system, several different standard sections may be established, depending on the type of construction to be used (compacted, semi-compacted, uncompacted for berms, or hydraulic fill). The use of standard sections is generally limited to levees of low to moderate height (usually less than 15 feet) in reaches where there are no serious underseepage problems, weak or unstable foundation soils, or undesirable borrow materials (very wet or very organic). In many cases the standard levee section has more than the minimum allowable factor of safety relative to slope stability, its slopes being established primarily on the basis of construction and maintenance considerations. Where high levees or levees on foundations presenting special underseepage or stability problems are to be built, the uppermost waterside and landside slopes of the levee are often the same as those of the standard section, with the lower slopes flattened or stability berms provided, as needed. When standard levee sections are applied, seepage and stability analysis must still be completed for the appropriate embankment and foundation material properties but the number of sections requiring analysis may be minimized.

7.2.1.3 When standard levee sections are locally adopted, stability and underseepage analyses are still conducted. However, when borings for a new levee clearly demonstrate foundation and borrow conditions similar to those at existing levees where good performance has been proven, such analyses may be very simple and made only to the extent necessary to demonstrate levee stability. In addition to being used in levee design, standard levee sections are often readily applicable for initial cost estimates and emergency and minor maintenance repairs.

7.2.2 Slopes.

7.2.2.1 Levee stability must be considered for all expected load conditions for new and existing levees. All levees require detailed seepage and stability analysis. The basis for determining the number of sections/levee reaches for analysis is discussed in Chapter 5. Seepage analyses techniques and requirements are detailed in Chapter 6.

7.2.2.2 Levees may not require extensive stability analysis if stability can be reliably documented from review of performance and analyses at a minimal number of sections (see Chapter 5 for further discussion on past performance and reach selection) for the following levee characteristics: (1) low to moderate height (generally, less than 15 feet) and (2) levees to be built of appropriate engineered material and resting on proven foundations. The relevance of past performance to future performance is dependent on the magnitude AND duration of past loading compared to the magnitude and duration of expected future loading. For levees where slope stability is not a controlling factor, the selection of levee slopes is controlled by practical considerations such as type and ease of construction, maintenance, seepage, and slope protection criteria. Some of the factors that need to be considered for all levees, but especially low levees where slope stability analysis might support steep side slopes, include the following:

- Type of Construction. Levees constructed with high compaction effort (see Chapter 10 for general compaction/construction categories) generally enable the use of steeper slopes than those of levees constructed with low compactive effort or hydraulic means. Space limitations in urban areas often dictate minimum levee sections requiring select material and proper compaction to obtain a stable section.

- Ease of Construction. A 1 vertical on 2 horizontal (1V:2H) slope is generally accepted as the steepest slope that can be easily constructed while ensuring stability of any riprap layers. When geotextiles are used beneath riprap layers, flatter slopes may be needed to maintain riprap stability along the geotextile/riprap interface.

- Maintenance. A 1V:3H slope is typically the steepest slope that can be conveniently traversed with conventional mowing equipment. Slopes steeper than 1V:3H may require specialized mowing equipment and designers should coordinate with the maintaining agency if they anticipate recommending slopes steeper than 1V:3H.

- Seepage. The location of the phreatic surface within the levee embankment often controls the stability of the landside levee slope. Sand levees are expected to reach steady-state through-seepage conditions for all floods; for such levees, a 1V:5H landside slope is typically considered flat enough to prevent damage from seepage exiting on the slope (Schwartz, 1976). For other levee materials, the landside slopes are selected based on past experience in conjunction with analysis and design where designers must consider the regional hydrology (loading duration) and the earth fill materials used in construction.

Generally, steady-state seepage conditions are assumed for levee and/or foundation materials when conducting initial design analyses. When conducting risk analyses, transient seepage analyses may also be considered together with sensitivity analyses to help assess factors contributing to instability, such as short-load duration in flashy watersheds where floods may rise and recede faster than steady state seepage conditions can develop. Other information should also be considered including but not necessarily limited to the following:

(1) Have existing levees been loaded to design flood conditions and what has past performance demonstrated?

(2) What is the calculated factor of safety if steady state-seepage conditions develop? Is the calculated factor of safety less than 1? Would instability lead to potential breach or exacerbate seepage or erosion problems?

(3) How well are the hydraulics and loading understood?

- Setback and Slope Angles for Erosion Mitigation. In coastal applications or areas with a wide fetch (such as bypass channels), designers may consider waterside slopes flatter than those required for stability as a means to mitigate erosion damage from wind-wave action. On riverine levees, designers may set levees back from the riverbank rather than place erosion protection in the channel at the time of construction. This practice allows further channel erosion to occur over time, without incurring immediate construction costs. If this is done, erosion limits need to be established so that erosion protection can be placed to mitigate continued erosion that could lead to overly steep slopes and undermining of the levee. As described in Chapter 4, a foreshore width of 200 feet or more is typically required to prevent migration of the river channel into waterside borrow areas. Proximity of the river channel to the levee toe, along with predicted erosion rate, are important considerations when establishing the setback distance of a levee. Additional measures for erosion protection, beyond slope flattening or levee setbacks, are discussed in Chapter 9.

7.2.3 Crown Width. The width of the levee crown depends primarily on roadway requirements and potential emergency response needs. Levees with the minimum crown width of 10 feet are typically not suitable for driving during a flood event for standard sized automobiles. Where there is a need to drive on the levee crown during flood events, a minimum 12 foot crown width is recommended. Narrower crown widths should not be used because of driver safety concerns and to not cause sloughing at edge of the levee crown during maintenance and flood fighting surveillance. Wider turnaround areas may need to be provided at specified intervals as discussed in Chapter 11. Crown widths greater than 12 feet may be required, depending upon the levee purpose and practice in a district. Where the levee crown is to be used as a public road, its width is usually established by the responsible transportation agency.

7.3 Effects of Fill Characteristics and Compaction.

7.3.1 Compacted Fills. The types of compaction, water content control, and fill materials govern the strengths of the fill and the steepness of levee embankment slopes if foundations have adequate strength. All fill material properties must be understood for levee design and controlled during construction so the levee will perform as expected.

7.3.2 Hydraulic Fill. Traditional and modern hydraulic fill consists mostly of pervious sands built with one or two end-discharge or bottom-discharging pipes. In the past, hydraulically placed clays have also been used for levee construction, primarily in the late 1800s and early 1900s, but this generally is not a modern construction practice.

7.3.2.1 Use of hydraulically placed material is not recommended unless the in-place properties can be confidently controlled and predicted for design, followed by field verification of characteristics at completion of construction. Depending upon techniques used during construction, hydraulically placed sands can range from low to relatively high densities (e.g., Sladen and Hewitt 1989, Lee et al. 1999). Tracked or rubber-tired bulldozers or front-end loaders are often used to move the sand to shape the embankment slopes and may cause some limited compaction.

7.3.2.2 Because levees constructed of hydraulic fill sand tend to be very pervious, they require a larger levee footprint to accommodate through-seepage during flooding (see Figure 7-1 and Schwartz, 1976). Hydraulic fill sands are susceptible to relatively rapid erosion from stream flow and would also quickly erode upon overtopping or where an impervious covering was penetrated. Because sand has low erosion resistance, the probability of breach associated with overtopping of sand levees is greater than is associated with clay levees. For that reason design features and anticipated intervention (flood-fighting) to prevent overtopping for levees with hydraulic fill sands are very important and typically required.

7.3.2.3 Seismic instability resulting from liquefaction has been associated with loose foundation materials and hydraulic fill and may be a problem (for example, Finn 1998), but often the likelihood of a damaging seismic event followed by flood that will cause inundation behind the damaged levee is low. In many locations, damage caused by levee foundation and embankment liquefaction can be repaired prior to the next damaging flood event, such as reported for the 1989 Loma Prieta event (for example, Pajaro River levee, Holzer 1998 and Miller and Roycroft 2004), as well as in Japan following the Kobe 1995 earthquake and the 2011 Tohoku-oki earthquake and tsunamis. If the likelihood of a damaging earthquake and flooding soon afterward are both high, then detailed risk analyses, including estimates of time and materials necessary to make post-earthquake repairs, may be warranted to assess the need of pre-seismic event slope instability mitigation.

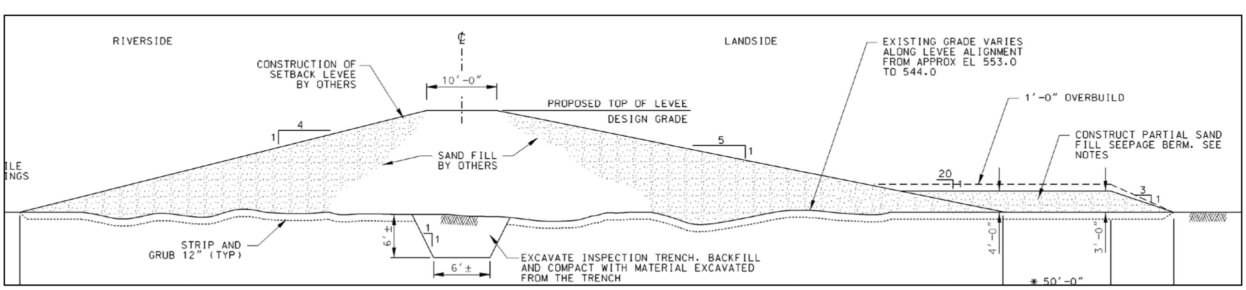

Figure 7-1. Sand Fill Levee (Iowa River Flint Creek Levee Repairs, CEMVR).

7.4 Reach and Analysis Cross-Section Selection.

7.4.1 General. As discussed in Chapter 5, evaluation and design of flood risk management projects considers separate contiguous reaches of the project along the alignment. Reaches are defined such that each analysis cross-section represents relatively uniform and consistent geometric and levee/foundation conditions within that reach (see Section 5.4). The analysis cross-section used for evaluation or design is prepared at the location within the reach that corresponds to the expected most-critical conditions. The number of reaches required for design varies between projects depending on uniformity of topography and subsurface conditions. The number of reaches can also vary depending on the level of study (for example, a reconnaissance level report will have less detail and fewer reaches than the same project at final design). The critical analysis cross-section for slope stability may differ from the critical cross-section for seepage, but stability and seepage analyses are often performed on the same cross-section where the pore-water pressures from a finite element method (FEM) or blanket theory seepage analysis are imported into a corresponding slope stability model. Applying FEM seepage results with slope stability analyses is not a design requirement, so it is not always necessary to perform seepage analyses at every stability analysis section, but with modern computational tools there is often little extra effort needed to complete the computer seepage analysis.

7.4.2 Geometry. Although it is common to maintain a consistent levee configuration along the project, changes in ground surface elevations (i.e., existing ground level and levee heights) and levee alignments near channel banks and landside ditches/canals can affect levee stability and the required levee section configuration. In general, higher levees and weaker foundations are more likely to be critical. For critical sections, evaluations and design should be based on actual topography and bathymetry.

7.4.3 Foundation Characterization. Levee foundations can range from relatively uniform to complex. Consequently, the required number of design reaches is variable and the designer must select the minimum needed to spatially characterize the foundation (see Chapter 5 for subsurface interpretation and reach selection).

7.5 Shear Strength Selection.

7.5.1 General.

7.5.1.1 Shear strength is defined by Duncan et al. (2014) as, “the maximum shear stress that the soil can withstand.” The proper assessment of shear strength for slope stability analyses is a critical aspect of understanding and predicting levee slope stability performance. Discussion of shear strength characterization and how it applies to slope stability is covered in EM 1110-2-1902 Chapter 2 and Appendix D. The information from those sections of that EM is generally applicable for levee and embankment dam evaluation, with the exception of soft soil undrained response during flooding, which is an important consideration for levees.

7.5.1.2 The selection of shear strength for evaluation of levees must address uncertainty in strength properties assumed for the stability analysis, and the sensitivity of the outcome to variation in the strengths. Large coefficients of variation for data may be “…in part because these test data many times reflect small-scale variations in soil properties, and in part because the tests themselves may introduce significant measurement error" (Baecher and Christian 2003). Measurement error is due to biases related to issues such as sample disturbance, which often (though not always) tends to reduce soil strength, laboratory test procedures, or model bias where conservatism can be introduced in the model used to represent the soil strength (e.g., linear versus curved failure envelope, especially at low stresses). Random measurement error is the scatter in data due to instrument or operator variability, and it is often assumed to distribute equally (Baecher and Christian 2003). Some also believe that the wide variation in properties is due to improper interpretation of loading conditions and associated expected soil behavior and is thus not attributable to the soil, but rather the analyst.

7.5.1.3 When using probabilistic methods to evaluate the stability reliability of levees, expected material properties (means or medians) with associated distributions are often used in probabilistic computations to develop estimates of likelihood of instability and failure as a function of load magnitude.

7.5.1.4 When conducting deterministic initial design analyses, the factors of safety presented in this manual will continue to be based on the inherent conservative bias often used by the USACE in selection of soil shear strength, referred to as the 1/3:2/3 rule: the soil shear strength is selected so that roughly 1/3 of the appropriate in-situ and laboratory testing data points are lower than the design shear strength and 2/3 are higher; the initial design strengths would thus be at approximately the 33rd percentile of their distribution and thus more likely to be conservative.

7.5.1.4.1 Engineering judgement is required by designers when developing initial design strengths. There is sometimes outlier laboratory testing that may need to be excluded when there is sufficient justification. Depending on the field and laboratory testing program, design shear strengths that are developed following the 1/3:2/3 rule may be based laboratory testing, in-situ testing, or a combination of in-situ and laboratory testing. The reliability of each of the techniques used to develop the design shear strengths should be considered.

7.5.1.5 The application of the 1/3:2/3 rule is often not appropriate for the selection for unit weights for initial deterministic analysis. The use of median unit weights for each soil strata is appropriate for most stability analysis. There are case histories where lower unit weights were applied in engineering analysis in an attempt to be "conservative", but actual unit weights in the field were much higher which resulted in lower factors of safety. Likewise, the assumption of higher unit weights in soil strata within features such as stability berms may result in an unconservative design if actual unit weights in the field are lower. Selecting unit weights above or below the median expected value to be conservative are only appropriate when designers have a clear understanding of the impact of that change on the critical slip surfaces.

7.5.2 Generalized Stress-Strain-Strength Behavior - Critical State Soil Mechanics.

7.5.2.1 The concepts underlying critical state soil mechanics (CSSM) are useful for understanding soil peak states, as well as how soil shear strength changes due to (i) drainage (or dissipation of pore pressures); (ii) softening; (iii) construction activities; and (iv) environmental loading conditions. Section D-7 in EM 1110-2-1902 describes how an understanding of the stress-strain response of soils is useful in interpreting the results of laboratory shear tests. Shewbridge and Schaefer (2013) outline how application of these concepts can be used to interpret soil strength for assessing dam and levee slope stability. Holtz et al. (2011) provides a succinct summary of CSSM and how it relates to conventional interpretation of laboratory tests, assessment of strengths in geotechnical analysis, and development of advanced constitutive models for use in monotonic and dynamic loading deformation analyses.

7.5.3 Evaluation of Drained and Undrained Strength. A range of methods may be used for selecting and assigning shear strength properties to levee embankment and foundation materials. The methods range from estimating strengths using empirical relationships (related to simple index testing) to comprehensive in-situ and detailed laboratory shear strength testing (Chapter 3) combined with careful evaluations of the full range of soil behavior over the range of potential loadings. Published relationships may be and are often used for preliminary analyses, but advanced design and risk analysis projects may warrant site-specific testing. The following paragraphs describe in more detail how drainage conditions affect the strength of soil and suggest how stress-strain characteristics influence shear strength parameter selection.

7.5.3.1 Undrained Strengths.

7.5.3.1.1 Undrained shear strengths are typically assigned to fine-grained soils that are loaded faster than excess pore pressures generated by consolidation and shear can dissipate. At this time, there is no widely accepted method for evaluating these excess shear induced pore pressures (see Johnson 1975, Duncan et al. 2014, Holtz et al. 2011, and Bishop and Bjerrum 1960 for more information). Such conditions typically occur during or at the end of construction, where little time has passed, so no consolidation and corresponding decrease in void ratio occurs that would produce an increase in shear strength in saturated soils. In this case, the shear loading comes from the placement of the embankment fill. Relatively rapid loading can also occur during flooding. In these instances the embankment may or may not have time to consolidate under its own weight and other forces acting on it; the undrained shear strength must reflect the degree of consolidation. Normally consolidated soils that have not yet consolidated under in-situ soil conditions are sometimes referred to as “under-consolidated.” Finally, undrained strengths may also control during very rapid loading of even relatively coarse free-draining soils, such as during seismic shaking, when pore pressure generation can lead to liquefaction.

7.5.3.1.2 For dense coarse-grained soil or over-consolidated fine-grained soil, undrained strengths will be very high, long-term drained strengths will be lower and control slope stability assessment. In contrast, if coarse-grained soil is loose or fine-grained soil is normally to slightly over-consolidated, the undrained strength will be lower than the same soil under drained conditions (Figure 7- 2 and 7-4). Conceptually, undrained conditions can be modeled using effective stress parameters, but pore-water pressures generated during shear must be estimated and are often too difficult to reliably model (Johnson 1975 and VandenBerge et al. 2015). Instead, undrained strengths, estimated based on effective stress conditions prior to the applied load condition, are routinely used for fine-grained soils in staged- and end-of-construction and other rapid loading cases, including floods.

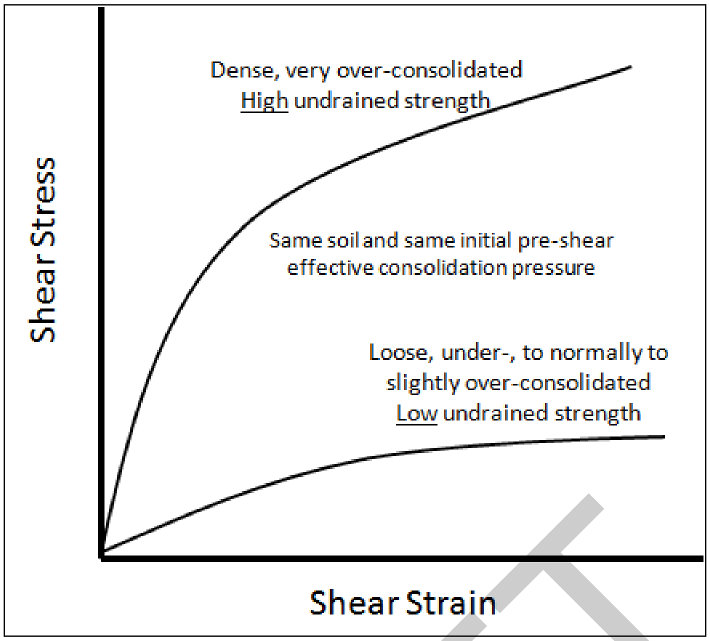

Figure 7-2. Generalized Undrained Stress Strain Curves for Loose and Dense Soils.

7.5.3.2 For fine-grained soils that were normally- to slightly over-consolidated prior to embankment construction, increases in mean effective stress due to consolidation will increase undrained strength, similar to the results of a stage construction analysis (e.g., Ladd 1991, Duncan et al. 2014). The soil undrained strength will vary horizontally and vertically under the embankment, which needs to be accounted for, both in site investigation as well as selection of strengths for analysis. The strength under the crown will be higher than free field strength in front of the toe (e.g., Leroeuil 2001, Duncan et al. 2008). To account for this, undrained soil strength for analysis of an existing embankment will often require the embankment to be broken typically into multiple sections. An example of this concept is shown in Figure 7.3

7.5.3.2.1 In each section, strength should be measured, through sampling and laboratory testing or through in-situ testing, at a minimum along the centerline (higher strength) and in front of the toe (lower strength) and modeled using different strength versus depth profiles in each zone under the embankment. Note that these undrained strengths are dependent on the degree of consolidation achieved at the time of loading (e.g., 100-year flood 15 years after construction) and may change with the passage of time. Thus, fine-grained foundation soil strengths during floods immediately after construction may be lower than soil strengths during floods occurring decades later after significant consolidation has occurred. In contrast, if shear loading is applied slowly enough for much, but perhaps not all the shear induced pore pressures to dissipate prior to rapid shear loading, then more sophisticated soil strength analyses, such as anisotropic consolidated strength assessments, may be appropriate.

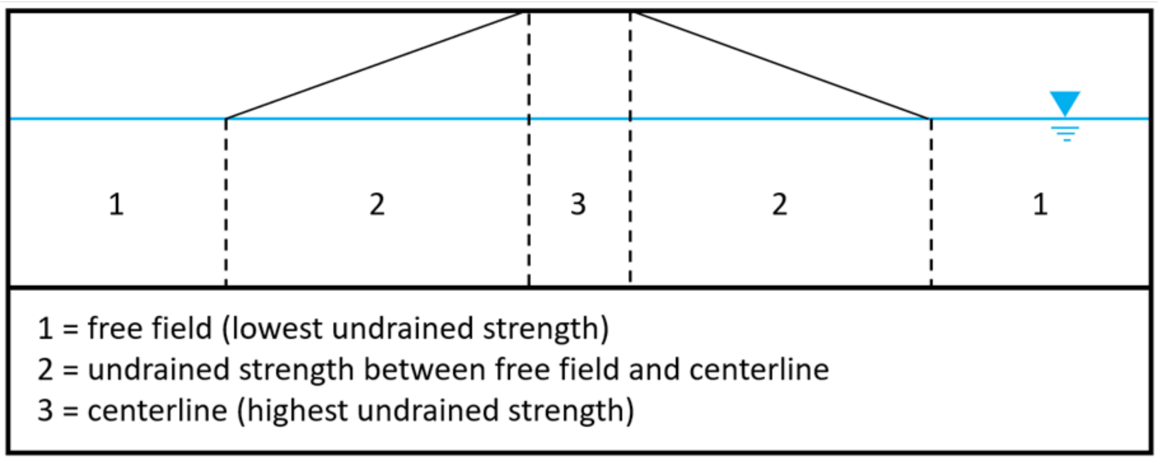

Figure 7-3. Simplified Zones of Consolidated Undrained Strength below an Embankment.

7.5.3.2.2 Staged construction with a preload or surcharge is sometimes used as a means of minimizing the levee footprint and reduce fill quantities in areas with saturated, soft, and compressible soils. The concepts outlined in Section 7.6.2 can be employed to assist in the development of strength gain estimates during staged construction. Designers that are considering the use of staged construction will need to have a clear understanding of the stress history at the time of sampling of any field testing/sampling (CPTs, field vanes, borings, etc.) and associated laboratory testing, an estimate of the time rate of settlement of each compressible layer under the preload or surcharge, and the degree of consolidation of each layer over time. This information can be evaluated to estimate future shear strengths over time. There is always uncertainty in estimating both total settlement and time rate of settlement in highly compressible soils. Soil parameters of each compressible layer, such as the degree of consolidation (Cv), can be critical in developing strength gain estimates and should be determined as part of the field and laboratory investigation program. Even with high quality laboratory data there is often scatter in the test results from consolidation tests and there are often fewer consolidation tests on most projects, consequently it is important for designers to consider a range of potential compressibility parameters and to understand the project impacts over this range. There are also often smaller coarser grained seams and layers that accelerate consolidation of individual clay layers that often impact the degree of consolidation of individual layers that are difficult to predict. For these reasons, staged construction shall be accompanied by an appropriate field and/or laboratory strength verification program during construction. Staged construction is further discussed in this manual in Sections 8.6.2, 8.6.3, 8.6.4, 10.4.4, and Appendix H.

7.5.3.3 Drained Strengths.

7.5.3.3.1 Under static loading, most soil materials that consist of clean, high hydraulic conductivity sands and gravels can be assumed to drain during compression and shear, and their shear strength is a function of effective confining stress and friction angle. During most typical loadings, including a fast-rising river or hurricane storm surge, these higher hydraulic conductivity materials are expected to drain. Therefore, any shear-induced excess pore-water pressures will dissipate nearly instantaneously in these soils. Excess pore pressure in clays may also dissipate if loading is slow enough that excess pore-water pressures generated by compression or shear dissipate, leading eventually to a non-transient (steady-state) seepage condition and anisotropic consolidation. It should be noted that loadings by nature can rarely be expected to be applied at the precise rate necessary for shear-induced positive pore pressure drainage and consolidation to occur, so lower undrained strengths may still control.

7.5.3.3.2 Drained strength is generally expressed using effective stress parameters, so an estimate of expected pore-water pressures is required. The pore pressures can be represented in stability analyses by piezometric surfaces or by a field of pore-water pressures estimated from seepage analysis by flow net or finite-element seepage analysis, often supplemented and verified by any piezometric measurements and seepage observations that are available. For initial design analyses, these pore pressures are typically based on steady-state seepage analyses. Though they are beginning to be applied for research and forensic studies (e.g., Stark et al. 2017), transient seepage results are not yet considered robust and reliable enough for routine design (Section 7.6.1.3). Transient seepage analyses can be used in parametric studies, where changes in an analysis result are assessed to aid in the risk-informed evaluation and design process.

7.5.3.3.3 Strain Hardening, Strain Softening, Peak, Fully-Softened, Post-Peak, and Residual Drained Shear Strengths. Drained strengths are affected by numerous factors discussed in the paragraphs below, which are correlated with the conditions of soil strength and deformation portrayed in Figure 7-4.

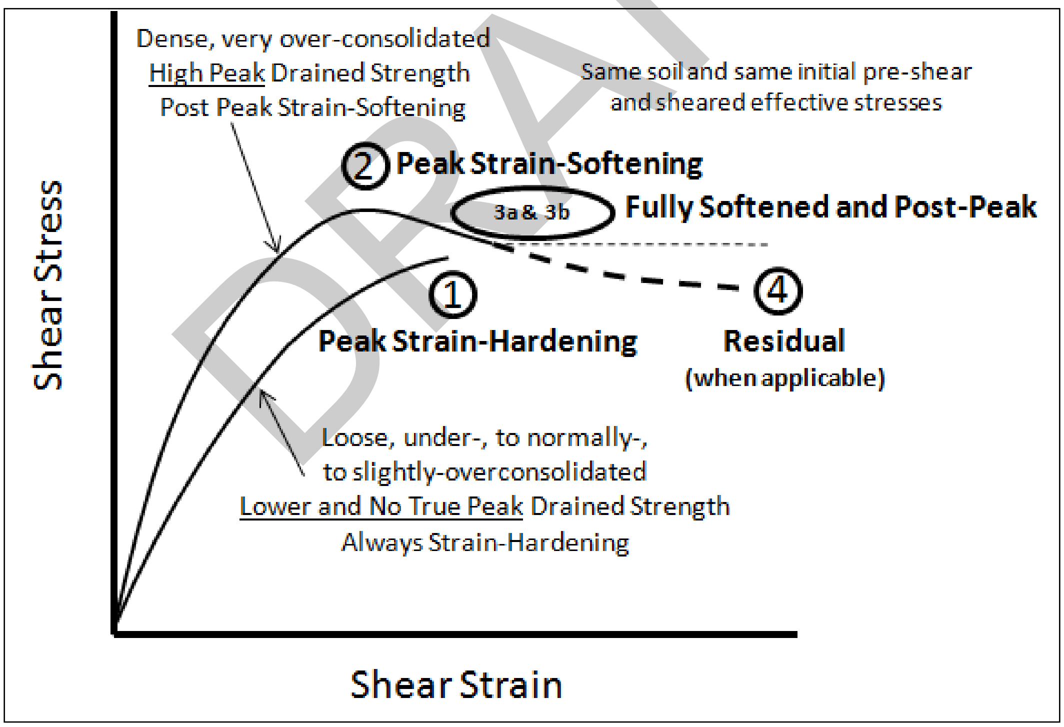

Figure 7-4. Generalized Drained Stress Strain Curves for Loose and Dense Soils and Types of Strengths Considered for Use in Stability Analyses. Depending on soil minerology, both Peak Strain hardening and Peak Strain-Softening materials can follow either "dashed" stress strain paths. See Skempton (1985) for more details.

7.5.3.3.3.1 Peak Shear Strength for Relatively Loose Strain-hardening Soils. For materials that exhibit strain hardening with no post-peak softening (for example, loose to medium dense sands and very slowly-loaded under-, normally-, and slightly over-consolidated fine-grained soils, such as laboratory testing of reconstituted clays to assess fully softened strengths, to be discussed below), peak shear strengths are generally controlling and are recommended for use in stability analyses. See “1” on Figure 7-4.

7.5.3.3.3.2 Peak Shear Strength for Relatively Dense Strain-softening Soils. For materials that do exhibit strain softening effects and that will not be loaded to shear stresses greater than peak, peak shear strength can be considered in slope stability analysis. Such materials include dense sands that are denser than their critical state, some well compacted and/or desiccated lean clays (i.e., soils with little to no volume changes during climate and moisture changes), and materials not subject to creep and progressive failure. These materials typically experience little change in void ratio with age and the peak shear strength is less likely to degrade significantly with time. See “2” on Figure 7-4. Analysts are cautioned, however, that if peak strengths are exceeded in any part of the slope, strain softening behavior can result in strain incompatibility where peak strengths may be achieved and surpassed along part of the slip surface and not attained or surpassed along other parts, such as occurs during progressive slope failure. Where strain softening or progressive failure can occur, fully softened, post-peak, and residual strengths, described in the following paragraphs, may be more appropriate for use in computations.

7.5.3.3.3.3 Fully Softened Shear Strength for Strain-softening Soils. For materials that do exhibit strain softening effects and that may be loaded to shear stresses greater than peak shear strength, including slopes vulnerable to progressive failure, the analyst should consider whether fully softened shear strength (FSS) should be used in slope stability analysis. FSS is a concept introduced by Skempton (1970) and is complimentary to the critical state soil behavior model described above. As reported by Duncan et al. (2011a):

> Skempton suggested that “we may say that the fully softened strength parameters c' and ϕ' are equal numerically to the peak strength parameters of the normally consolidated clay. Equating the [drained] strength of normally consolidated test specimens to the “fully softened” strength in this manner is a somewhat conservative approximation. This observation should be viewed as an empirical conclusion rather than a fundamental principle of soil behavior.”

7.5.3.3.3.4 FSS is typically used to represent long term drained shear strength conditions of stiff fissured clays and shales and compacted fat clays and are represented with drained material properties modeled in terms of effective stress. It is typically used to assess the factor of safety for “first time” slides in soil and soft rock slopes that have not yet developed distinct large strain failure surfaces (e.g., Skempton 1970). See “3a” on Figure 7-4.

7.5.3.3.3.5 An FSS testing standard is available for the ring shear device in ASTM D7608 and a suggested test method for the direct shear device can be found in Stephens and Branch (2013). Correlations of FSS friction angle and liquid limit are also available (Stark et al. 2005). The triaxial apparatus can sometimes also be used to assess fully softened shear strengths, but no widely-accepted standards are available at this time. The concept of FSS has been applied and is generally accepted as an analysis approach for assessing long term performance of cut slopes, after weathering and aging, and has also been applied to levee slope evaluation, particularly for fat clays subject to significant desiccation and flood rewetting, often experienced in arid environments. Weathering processes of wetting and drying (e.g., Wright et al. 2007 and Take and Bolton 2011) and freezing and thawing result in volume changes and micro-straining of expansive materials. Levees constructed of expansive fat clays are subject to greater cyclic volume changes and may form cracks and fissures (both parallel and normal to the embankment slope) with associated accelerated aging and environmental changes. In this way the peak shear strength in levees is reduced to a lesser value than at the time of placement and compaction for these over-consolidated soils. Surface sloughs, and in some cases relatively deep slides (compared to the height of the levee), can result following extended dry periods followed by significant rainfall. Desiccation cracking can lead to more rapid saturation of an embankment or slope than non-desiccated conditions and further contribute to slope instability. Typically, maintenance-type slides can be repaired prior to flooding, but prolonged extensive rainfall and subsequent flooding are not mutually exclusive events and any slide that occurs during flooding can be problematic. Compounding the loss of strength from weathering, levees that experience desiccation cracking are subject to leakage through transverse cracks during flooding as the cracks may or may not fully heal. Open desiccation cracks were experienced during the 1997 flood of record in Grand Forks, North Dakota, and leakage through the cracks had to be addressed during flood fighting (Schwanz 2015, personal communication). To model these effects, often more representative slope stability analysis results can be achieved using a curved failure envelope to describe the high sensitivity of FSS strengths to low stress levels, helping to avoid trivial and often unrealistically low shallow-slide factors of safety (Wright et al. 2007).

7.5.3.3.3.6 Post-peak or Ultimate Shear Strength for Strain-softening Soils. As discussed above, the FSS concept provides a means of assessing and assigning long term shear strengths to over-consolidated soils, such as high plasticity clays subject to weathering, creep, and progressive type failure. Unfortunately, FSS testing is not routine in many laboratories and further, without established standards for performing FSS testing using conventional triaxial or direct shear apparatuses, analysts may not have a readily available method to assess FSS strengths.

7.5.3.3.3.7 In some situations, time and cost constraints have led to using other more common standard tests to estimate FSS using measured post-peak shear strengths. Post-peak shear stress, as the name implies, represents the shear stress measured at strain or displacement increments during continued loading after the maximum shear stress in the specimen has developed. Because there is not a stress state associated with a physical condition, it is difficult to choose, with consistency, a single value for analysis strengths. To approximately estimate strengths and to address concerns of progressive failure and strain compatibility along potential slip surfaces, as well as the propensity for creep, the selection of shear stress associated with strain past peak has been used when performance has been verified with local practice (Lefebvre 1981 and Schwanz 2016 personal communication USACE Red River Projects). See “3b” on Figure 7-4.

7.5.3.3.3.8 Residual Shear Strengths. Residual shear strength is used to characterize materials that have already undergone failure and large strains along distinct sliding surfaces. Such a condition may occur where an embankment is constructed near or on an existing slide in a clay foundation, or when excavation unloads the toe of a slide. Particularly for clayey material, with plate-shaped particles, the large strains can cause reorientation of particles so they slide over each other more easily, without dilation or contraction. (See "4" on Figure 7-4.) In plastic clays, the residual friction angle can be a small fraction of the peak friction angle. Residual strengths should be assigned to failure surfaces in slopes with previous slides or other geologic features that have experienced large strains in the past, leading to the development of a distinct failure surface. Residual strength can be measured in the ring shear device per ASTM D6467. One of the best methods to assess residual strengths is back-analysis of failed slopes, especially those that have experienced large movements. Back-analysis requires good estimates of the pore pressures that existed at the time of the slide, and accounting for any significant 3D effects. The results of back analysis can be used in conjunction with laboratory shear testing, most commonly non-ASTM "reversal" direct shear tests, which are discussed in EM 1110-2-1906. If the position of a preexisting failure surface is known (from inclinometers or field observation), then this position should be used in the back analysis or, as an alternative, a thin zone of material at the residual shear strength can be used that follows the slide rupture. (This does not mean that other sliding surfaces can be ignored.) If the position of the failure surface is not known, a search procedure should be used in both forward and back analyses.

7.5.3.3.3.9 Residual shear strength properties are typically expressed as drained shear strength properties in effective stress analyses. Because residual shear strengths are the lowest possible, they are conservative and is rare to use them for design unless there is a pre-existing slide plane or troublesome foundation layer. See Chapter 1 for more information on seeking a deviation from typical design standards. When reconstructing failed slopes it is recommended that the failure masses be excavated back into undisturbed material, where possible, to remove the weakened failure surface and therefore remove the need to design for residual strengths of the sheared materials.

7.6 <u>Pore Water Pressures and Associated Strength Definition Methods for Analysis</u>.

7.6.1 <u>General</u>.

7.6.1.1 As discussed above, strengths used in slope stability analyses are often described in terms of total stress parameters for undrained shear strengths and effective stress parameters for drained shear strengths; some analysis conditions may require a mix of undrained total and drained effective stress strength models, depending on the material types, configurations, and densities of the embankment and foundation materials, and the speed and duration of the loading condition. Pore pressure (water or air) is implicitly addressed in the testing and selection of undrained shear strength properties used in total stress analyses, but must be explicitly expressed for drained effective stress strength assessments. While conceptually valid, matric suction and pore-air pressures are typically ignored (compressibility of air is assumed not to affect inter-granular stress and pore-moisture suction is difficult to reliably predict) when assessing drained effective stresses for design, but may be appropriate for risk analysis purposes. In contrast, positive pore-water pressures, or changes in pore-water pressures, must almost always be accounted for. Positive pore-water pressures and changes can originate from several sources such as:

- flood loading;
- naturally existing groundwater or long term seepage conditions;

- fill placement or loading;
- excavations;
- poor surface water control;
- rainfall infiltration;
- broken water lines;
- and other loading conditions that change horizontal and/or vertical total stresses.

7.6.1.2 Negative pore-water pressures that are associated with matric suction (difference in pore air pressure, u_a and pore water pressure, u_w) from capillary action are sometimes considered in forensic and risk evaluations. It has been observed that matric suction does not increase soil strength in the same way as the net stress (σ-u_a), and a modifier is generally used on the matric suction component to better estimate strength (i.e., ϕ^b, Fredlund et al. 1978, Duncan et al. 2014).

7.6.1.3 Computer analysis tools to perform transient seepage analyses are increasingly available, but the ability to measure or otherwise evaluate parameters needed for unsaturated soil mechanics is not sufficiently established for initial design use and is still in question for risk analysis (VandenBerge et al. 2015). Transient seepage analysis tools do not yet have a proven track record, with calibrated model results compared to field performance case histories on levees. It is also an important point to make that transient analysis results are heavily influenced by site specific soil water characteristic curves and seasonal water content variations through the levee and foundation soil profile. In-situ instrumentation to understand transient flow through levees is not typically available. As such, transient results are not yet considered robust and reliable for routine design and may not always be appropriate for the final design and evaluation. Nevertheless, they can be effectively used to evaluate sensitivity to the parameters affecting saturation and development of pore water pressures and can help guide the analysts to a better-informed opinion about factors affecting performance and strength (e.g., Stark et al. 2017). If transient analysis is being used to inform the design on a project then site specific unsaturated soil parameters and seasonal water content variations will need to be determined.

7.6.2 Total Stress Methods to Evaluate Undrained Soil Strength.

7.6.2.1 When using total stress methods to evaluate saturated undrained soil shear strengths (i.e., when the undrained strength is set equal to the undrained cohesion and the undrained friction angle is set equal to zero), changes in the total stress theoretically do not affect shear strength of fine-grained soils. Therefore, pore-water pressure is often denoted as having a value of zero or it is explicitly not used to assess soil total stresses, depending on the input recommendations of the analysis software package used. Nevertheless, software packages do vary and the analysts should confirm that proper pore pressure and associated strength evaluations are being conducted.

7.6.2.2 Typically, undrained strengths for normally to slightly over-consolidated fine-grained soils is measured though a combination of in situ cone penetration and/or vane shear tests, which is supplemented by laboratory strength and oedometer tests. The developed shear strength profile based on lab and in situ test data are evaluated by first assessing the pre-loading (e.g., pre-flood) effective stress and preconsolidation profiles, which is then used to estimate undrained strengths available during loading using appropriate undrained strength models, such as the stress history and normalized soil engineering properties (SHANSEP) model (Ladd and Foott 1974 and Ladd and DeGroot 2003) or s_u/σ_c' (Duncan et al. 2014 and Terzaghi et al. 1996), accounting for various factors, such as anisotropic consolidation, aging, and applied shear rate effects, when appropriate.

7.6.2.3 The mode of shearing, i.e., anisotropically consolidated undrained triaxial compression (CAUC), anisotropically consolidated undrained triaxial extension (CAUE), or direct simple shear (DSS), needs to be accounted for when developing shear strength profiles. If failure surface orientation anisotropy is not directly modeled, it is recommended that analysts use the direct simple shear mode of failure as an "average" strength along potential failure surfaces when analyzing levees and embankments (i.e., Ladd 1991, Duncan et al. 2014). Undrained strength in the direct simple shear mode of failure tends to be between 60 and 100 percent of anisotropically consolidated undrained triaxial compression strength, as shown in Figure 7-5. Increases in the ratio of DSS strength to anisotropically consolidated triaxial compression strengths have generally been considered to increase with plasticity index (Larsson 1980, Mayne 1985, Ladd 1991), however, recent data from New Orleans (Brandon et al. 2011) shows a relatively constant ratio of DSS to CAUC normalized strengths. Due to uncertainty in mode of shear on undrained strength ratios, performing both undrained triaxial compression tests with pore pressures measurements and DSS strength tests (for the same tube sample) as part of a strength characterization program is often advisable, especially in locations where there is little past levee construction experience to support engineering judgment.

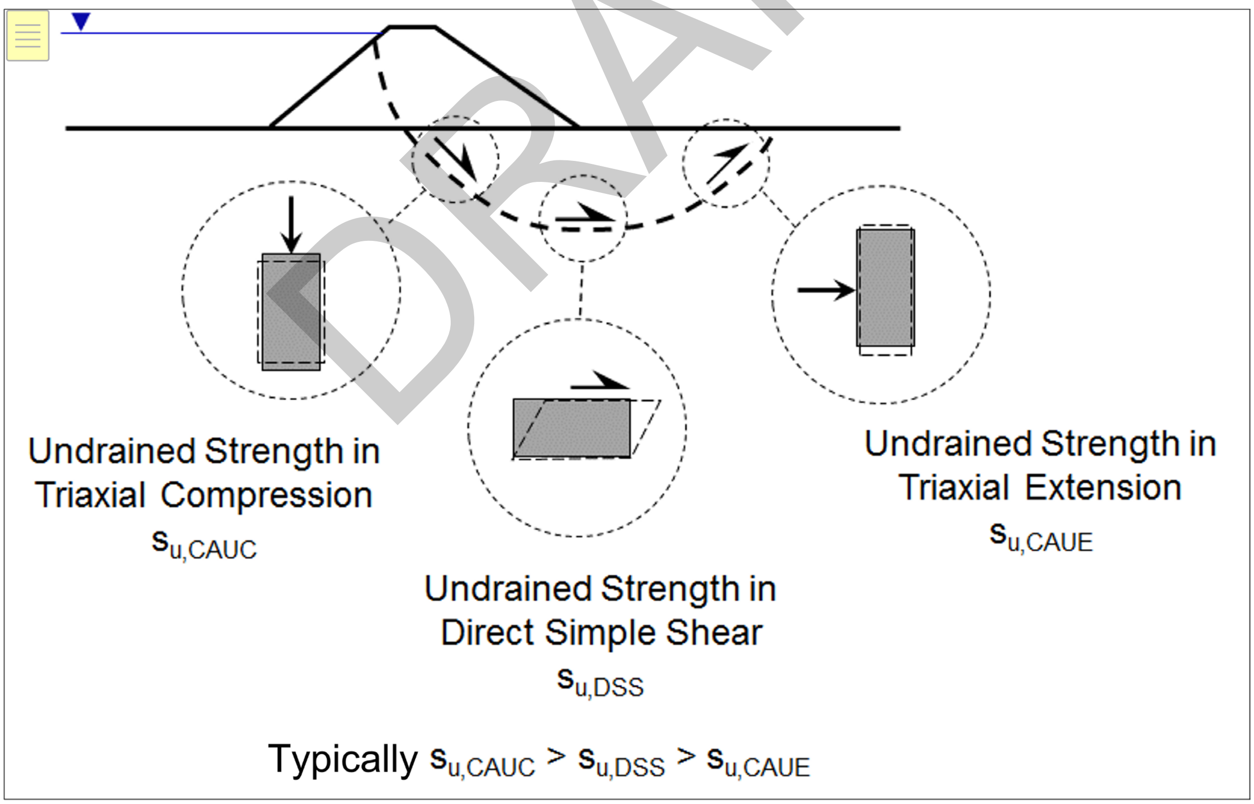

Figure 7-5. Modes of Shear Strength under an embankment (after Ladd 1991)

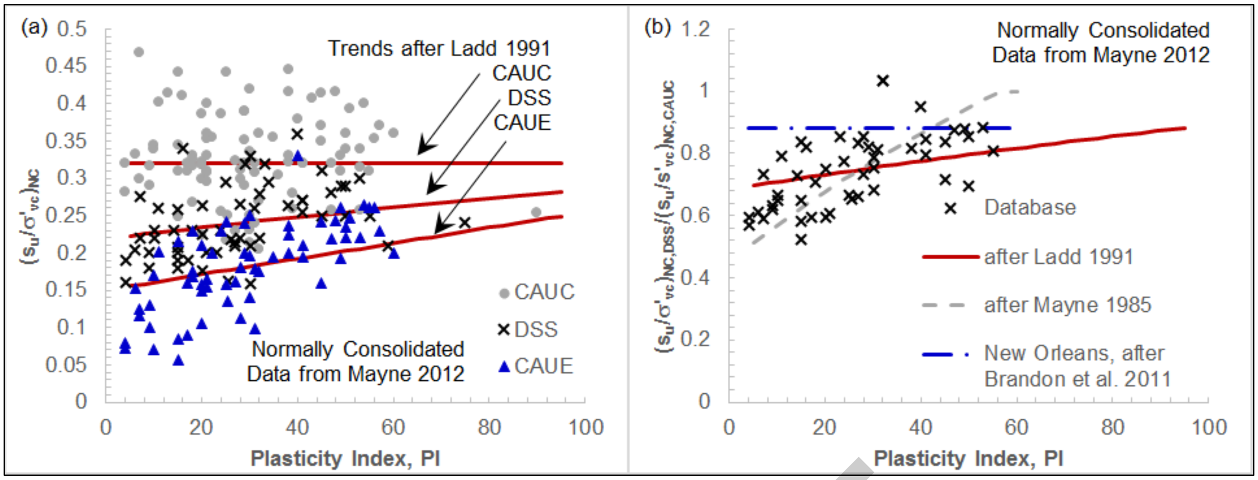

Figure 7-6. (a) Normally consolidated undrained strength ratios as a function of plasticity index for various modes of shearing; and (b) ratio of DSS normally consolidated undrained strength ratio to that from CAUC triaxial tests as a function of plasticity index

7.6.2.4 Typically, undrained strengths are not input into the analysis program with a total stress failure envelop; instead, undrained strength profiles which are functions of location, depth and time (for materials subject to rebound or consolidation) are defined for appropriate materials in the embankment and foundation.

7.6.2.5 Sometimes soils that are partly saturated may develop increased shear resistance as a function of an increase in total stress (i.e., total stress cohesion > 0 and total stress $\phi > 0$) and may also be modeled using total stress parameters. Again, pore pressures set equal to zero are still often explicitly specified in the stability analyses to avoid computational errors. More often, though, use of simple drained friction angles and curved failure envelopes yields more reliable and representative results.

7.6.2.6 Free draining soils within a total stress analyses use drained, effective stress shear strengths in the analysis and require pore-water pressures. Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady state seepage analysis techniques (piezometric lines, flow nets, finite element/difference analyses, etc.)

7.6.3 <u>Effective Stress Methods to Evaluate Drained Soil Strength</u>.

7.6.3.1 Pore-water pressures need to be defined when using effective stress methods to evaluate drained soil shear strengths. Common options for defining pore-water pressures include the following (several are commonly included in slope stability software):

- Using the phreatic surface (water table) as a piezometric surface and computing the pore-water pressure as the vertical distance from the piezometric line to the point of interest, multiplied by the unit weight of water. Strictly speaking, a single piezometric line is only correct if there is no vertical seepage gradient, although it is often a reasonable approximation.

- Specifying a set of piezometric lines in an aquifer from piezometer data and/or a closed form solution (that is, blanket theory often used in levee evaluations).

- Specifying pore-water pressure based on a graphical flow net by identifying pore-water pressures at points and interpolating between those points as needed.

- Importing pore-water pressures from finite element/difference solutions.

- Specifying pore-water pressures based on field instrumentation by identifying pore-water pressures at points and interpolating between those points as needed.

- Applying a pore-water pressure coefficient r_u (an older and generally outdated method, rarely used, but may occasionally be employed for evaluating clay embankments with rainfall infiltration using FSS where $r_u \leq 0.6$).

7.6.3.2 Using a phreatic surface or a single piezometric line to define stability model pore-water pressures has been successful on many projects and remains a reasonable approach. However, it can lead to unconservative results in the form of higher computed factors of safety when there is upward flow of water near the embankment toe (Duncan et al. 2014; Perri et al. 2012). As several geotechnical software suites offer FEM seepage analysis tools that facilitate automatic import of pore pressures into limit-equilibrium slope stability analyses or coupled with stress deformation software, it is now relatively easy to perform stability analysis using robust steady state seepage pore-water pressure regimes and is preferred. That being said, uncertainty in horizontal and vertical hydraulic conductivities of blanket and underlying permeable layers can span an order of magnitude, having a large effect on the calculated pore water pressures and calculated slope stability factor of safety. Uncertainty in pore pressures due to hydraulic conductivity and other assumptions within FE models should be considered parametrically when selecting pore pressures for slope stability safety factor and risk analysis reliability computations.

7.6.3.3 Steady-state seepage conditions are assumed for initial design analyses; however, transient seepage may also need to be considered when conducting risk analyses and developing risk informed designs (see Chapter 1 for the process to seek a design deviation). When transient conditions are being considered, analysis assumptions will receive significant scrutiny and analysts will be hard pressed to meet the burden of proof for a design deviation from the typical steady state assumption. Often the cost to explore and identify all potential defects in the embankment and foundation that could often lead to significant violations of the assumptions will be prohibitive. An unidentified defect in presumed unsaturated clay, such as a sand lens or layer, variable clay properties and degrees of saturation, desiccation cracks, an abandoned utility pipe, or an animal burrow, may significantly reduce the time for saturation to occur. In cases where unsaturated conditions are being relied upon (e.g., Öberg 1995, Westerberg et al. 2014), monitoring of embankment performance is likely even more important than for embankments designed for long-term steady-state seepage conditions. In addition, large, long-duration floods in flashy watersheds (ones where discharges increase and decrease rapidly in response to precipitation) may be very infrequent, so there would be little opportunity to observe behavior with elevated flood levels to confirm that performance is consistent with analysis expectations, and to respond before there are negative consequences (i.e., it may not be possible to apply the "Observational Method"). If transient conditions are required for acceptable system performance and short-term unsaturated or high undrained strengths are used in the short-term stability analyses, significant contingency plans to identify and rapidly respond to unexpected problems may be required, particularly for high consequence systems, such as large high-hazard dams and urban levees (Shewbridge and Schaefer 2013).

7.6.3.4 Typically, drained strengths are input into the analysis program by defining an effective stress failure envelope and associated phreatic surfaces or fields for the appropriate materials. Peak or fully softened strengths, as appropriate may be considered. Residual strengths should be used where previous shear deformation or sliding has occurred.

7.7 Conditions Requiring Analysis.

7.7.1 Loading Conditions. The loading conditions that a levee and its foundation may be subjected and which should be considered in analyses are designated as follows: Case I, end of construction; Case II, sudden drawdown from full flood stage; and Case III, flood. Each case is discussed briefly in the following paragraphs and the applicable type of shear strength is suggested and summarized in Table 7-1. It is important for analysts to recognize that loadings can occur at various periods of time relative to construction, with varying degrees of consolidation or rebound achieved; thus foundation and embankment strengths will likely change with time and multiple combinations of conditions with different limiting strengths may need to be considered. For these reasons, there are cases where separate analyses should be completed for Case III using total stress (undrained shear strengths) and effective stress (drained shear strengths) conditions. The controlling total stress or effective stress condition should govern levee design in these cases.

7.7.1.1 Slope stability analyses should be completed for a range of flood loading conditions. At a minimum, analyses are required for the water levels defined in Chapter 1 and illustrated on Figure 1-3 as the As-Constructed (Top-of-Levee) grade and Design Water Surface Elevation.

7.7.1.2 The water level required for Case I is the Normal Water Level. The Normal Water level is defined as the median annual water level; that is, the waterside level or groundwater level, whichever is higher. For levees affected by tides, the Normal Water Level should be the higher of either the median annual water level or the mean high tide.

7.7.1.3 There may be other water levels of interest that designers should analyze for consideration during the final evaluation and design.

7.7.1.4 The annual chance of exceedance of each water level analyzed during design shall be determined and documented for each analyses cross section. This information will be necessary for completion the final evaluation and design of the levee.

7.7.2 Case I – End of Construction. 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. For construction involving excavation, excess negative pore-water pressure from shear and rebound may also not have had time to dissipate. The end of construction condition is applicable to both the waterside and landside slopes of levees.

7.7.2.1 For low-hydraulic conductivity embankment and foundation materials that would be loaded in an undrained manner and will not undergo consolidation or rebound, results from laboratory unconsolidated-undrained tests (UU), direct simple shear (DSS), and vane shear (VST) strengths are applicable to fine-grained soils loaded under this condition. CPT correlations to undrained shear strength can be used if correlated to UU, DSS, or VST results.

7.7.2.2 For fine-grained soils, effective stress changes from either rebound or consolidation will occur during and after construction and will result in potential strength gains and losses. For consolidating materials (e.g., materials loaded by fill), strengths are typically smallest prior to consolidation. For rebounding materials (e.g., materials unloaded by excavation), strengths are typically smallest after rebound. For materials that undergo significant stress changes, undrained strengths can be evaluated from consolidated-undrained tests, accounting for the amount of over-consolidation, such as in the SHANSEP method (Ladd and Foott 1974 and Ladd and DeGroot 2003). Analysts should use the effective stress condition that represents the lowest “limiting” undrained strength in initial stability analyses. In most situations, analysts may want to ignore the strength gain associated with the increase in foundation effective stress due to fill placement.

7.7.2.3 For relatively high hydraulic conductivity materials that will be loaded in a drained manner, results of in-situ tests are typically used to select peak friction angles. Stress dependence of peak friction angle can be verified through laboratory CD triaxial tests.

7.7.3 Case II – Sudden Drawdown. This case represents the condition whereby a prolonged flood stage or even normal stage saturates much of the upstream portion of the embankment and then the flood elevation reduces faster than the soil can drain. This can cause higher pore pressures by causing excess pore-water pressure to develop from undrained shear, resulting in waterside levee and foundation slope instability. For the selection of the shear strengths, see EM 1110-2-1902 for more information. The weight of the embankment may be sufficient that over-consolidated materials in the foundation would be loaded to new higher consolidation pressures. These materials would then behave as normally- to slightly over-consolidated soils that would generate positive excess pore pressures during shear (for example, Stark et al. 2017). A challenge in this loading condition is evaluating appropriate water surface elevation reductions for the drawdown analysis, which may be affected by normal hydrologic and hydraulic conditions as well as abnormal level changes due to unexpected events. When selecting the drawdown water level, the analyst may choose a Normal Water Level as defined in Section 7.7.1.2 or another drawdown water level associated with an elevation where there is a significant reduction in the slope of an expected hydrograph recession. Rarely does this loading condition lead to levee breach, but it may need to be considered in the risk assessment, especially for levees along floodways with a potential for frequent consecutive low and high water events and in locations where waterside slope failures could impede channel flow and raise river levels, or removing wave attenuation berms and other potential negative impacts that could lead to breach.

7.7.4 Case III – Flood. Flood loading applies when water levels on the flood side exceed the landside levee toe elevation. This load case can include the steady seepage condition when pore-water pressures from seepage fully develop and materials with high conductivity are assumed to behave in a drained manner (e.g., sand levees on sand foundations or a fine-grained blanket, subjected to vertical gradients, overlying a sand aquifer). But it also addresses the likelihood that levee and foundation seepage pore pressures may only partly equilibrate and/or shear-induced positive pore-water pressures of low conductivity normally to slightly over-consolidated materials may not dissipate. Because combined flood rebound and shear-induced pore-water pressures are difficult to estimate, the strength of the fine-grained soils are represented using undrained strength parameters.

7.7.4.1 For undrained analyses of fine-grained soils, effective stress changes from either rebound or consolidation will occur during and after construction and from flood loading shear and will result in potential strength gains and losses. For materials that undergo significant stress changes, undrained strengths can be evaluated from consolidated-undrained tests, accounting for the amount of over-consolidation, such as in the SHANSEP method (Ladd and Foott 1974, and Ladd and DeGroot 2003). For consolidating materials (e.g., materials loaded by fill), strengths are typically smallest prior to completion of consolidation. For rebounding materials (e.g., materials unloaded by excavation), strengths are typically smallest after completion of rebound. Analysts should use the effective stress condition that represents the lowest "limiting" undrained strength in initial flood loading stability analyses. Though not often considered in levee evaluations because of analytical complexity, partial anisotropic consolidation prior to flood loading may also increase undrained strengths and may warrant consideration when conducting risk analyses and forensic investigations.

7.7.4.1.1 For new levees, the critical undrained flood condition is a rapid flood that occurs shortly after construction is completed. Undrained shear strengths can be estimated based on the expected strength gain during construction, but in most cases construction strength gains are ignored unless there is strength gain verification during construction.

7.7.4.1.2 For an existing levee the critical undrained flood loading is based on a flood today with the existing undrained shear strengths. The current undrained loading will need to be determined for the existing levee using field and laboratory data.

7.7.4.2 For effective stress analyses of fine-grained soils, use steady state seepage pore pressures to estimate the lowest flood effective stresses and the associated lowest "limiting" drained strength in initial stability analyses.

7.7.4.3 For coarse-grained soils loaded in a drained manner, using steady state seepage pore pressures to estimate flood effective stresses represents the lowest "limiting" drained strength used in initial stability analyses. Stress dependence of peak friction angle can be verified through laboratory CD triaxial tests.

7.7.4.4 Both effective stress and total stress analyses may be required for Case III. Historically, only effective stress slope stability analyses were completed for levees for the Case III Flood loading cases. However, there are circumstances when total stress conditions under flood loading may be the more critical loading condition. This may be the case for levees that are founded on soft, saturated, normally consolidated to slightly overconsolidated fine-grained deposits.

7.7.4.5 Both total and effective Stress analyses are required for all new levees.

7.7.4.6 Total stress analyses is not required when evaluating and modifying existing levees when any of the following conditions are met:

- Fine-grained deposits are not present within the foundation.
- Fine-grained deposits in the foundation are characterized as overconsolidated and have undrained shear strengths (Su) greater than 1,000 psf.
- There is existing analyses and past performance that can be documented to demonstrate that the required factors of safety will be met.
- There have not been alterations outside of the footprint of the levee (i.e., such as drainage ditch excavations, etc.) that have not been evaluated.

7.8 Deterministic Evaluation and Design Criteria.

7.8.1 General. The minimum required factors of safety (FS) for the design conditions discussed above are shown in Table 7-1. Chapter 1 describes flood load cases, water levels, and top of levee designations.

7.8.2 New Levees. Newly constructed levees are untested and stability must be forecast for all loading conditions. Levees constructed on foundations containing fine-grained materials typically settle from foundation consolidation, and if normally consolidated, there can be an associated increase in foundation strength over time. Utilizing the increase in undrained shear strength with time for the design of new levees may be used if supported by reliability and risk assessment results.

7.8.3 Existing Levees. Existing levees may or may not have experienced flood loading and may or may not have completed consolidation/rebound since construction. Existing levees that have performed satisfactorily during significant flood events often have a greater reliability at that level of loading than new levees that are untested. The exception to this is levees constructed of high plasticity clays that are prone to cracking and developing slickensides during seasonal weathering or those that are significantly impacted by damaging vegetation, animal burrows, and progressive deterioration of seepage conditions, such as caused by multiple events that trigger internal erosion, where these factors can change seepage conditions leading to higher than expected pore-water pressures, thereby reducing stability. Aging of existing levees, and the detrimental effects that aging may cause, must be considered when evaluating the stability of existing levees. Existing levees constructed on foundations containing fine-grained materials may also have experienced different degrees of consolidation and strength gain since construction; stability analyses should be based on shear strengths applicable for the time and loading condition of the analysis.

Table 7-1. Minimum Factors of Safety – Levee Slope Stability.

	Applicable Stability Conditions and Required Factors of Safety (FS)		
Type of Slope and Loading	**Case I End-of-Construction**	**Case II Sudden Drawdown[b]**	**Case III Flood[c]**
New or Existing Levees			
Normal water levels	1.3	--	--
Design Water Surface Elevation	--	1.0 to 1.2	1.3 (Total Stress) or 1.4 (Effective Stress)
Top of Levee [d] (New: As-Constructed) (Existing: Final Grade)	--	Analyses Required	Analyses Required
Other Water Levels	--	--	--
Other Embankments and dikes[e]	1.3[f]	1.0 to 1.2	1.4

[a] Total stress (undrained analysis) analyses is required for Case I.

[b] Sudden drawdown analyses: FS = 1.0 applies to drawdown from DWSE levels when the DWSE is unlikely to persist for long periods (i.e., partial saturation occurs) preceding drawdown. FS = 1.2 applies to DWSE levels, likely to persist for long periods (i.e., full saturation occurs) prior to drawdown.

[c] Both Total stress (i.e., undrained analysis) and effective stress (i.e., drained analysis) may be required for Case III depending on foundation conditions. Minimum factors of safety required for both conditions, when applicable, are provided.

[d] Analyses of Top of Levee is required during initial deterministic design. The results of these analyses should be considered during the risk assessment performed during Phase 2 of design to determine whether the levee will meet the level of reliability expected for the project.

[e] Includes slopes that affect the stability of a levee, such as those which are part of cofferdams, retention dikes, stockpiles, navigation channels, breakwater, adjacent river banks, and excavation slopes.

[f] Temporary excavated slopes are sometimes designed for only short-term stability with the knowledge that long-term stability is not adequate. Special care is required in design of temporary slopes, which do not have adequate stability for the long-term (steady seepage) condition.

7.8.4 Riverbanks and Channel Slopes. Many levees are constructed near rivers and channels, and stability toward the waterside must be considered during non-flood periods. Though consequences of failure are generally small during non-flood periods, slopes must be designed so they are not a maintenance burden for levee owners or local sponsors. Additionally, remediation measures should be able to be implemented quickly if they are required for satisfactory performance during a flood event. Stability calculations may show low factors of safety but other factors that tend to (temporarily, but possibly for a sufficient duration) improve stability are often present but are typically neglected during initial slope design analyses. Soil suction, vegetal root reinforcement, evaporation, and transpiration are effects that tend to improve stability but are difficult to explicitly include in numerical computations. An example of negative pore-water pressures temporarily improving stability until they dissipate is described in Torrey (1988) and Torrey et al. (1988). This two-part report, prepared for the Lower Mississippi Valley Division, describes erosion that removes a portion of a granular deposit below water and the over-steepened slope stands for short periods of time due to negative pore-water pressures related to shear dilation. These negative pore-water pressures then dissipate leading to slope failure, which can be very large, removing substantial portions of the underwater waterside slope and undermining and removing the entire levee. These factors can be evaluated qualitatively and quantitatively when conducting risk analyses.

7.9 3-D and Verification Analysis.

7.9.1 3-D Analysis. Levee embankments are typically long and can generally be characterized as a plane-strain loading condition that is best represented by two dimensional (2-D) methods of analysis. As discussed in EM 1110-2-1902, in some complex situations, three-dimensional (3-D) limit equilibrium analysis methods may require consideration of the effect of 3-D slip surface shapes and 3-D concentration of seepage. 3-D methods of analysis may be useful for the back-calculation of the mobilized shear strength of soils in existing slides. Generally, 3-D methods are not recommended for use in design because of their limitations as described in EM 1110-2-1902; however, rigorous 3-D solutions have been developed for simple slopes using limit analysis (Michalowski and Drescher 2009) and 3-D FEM solutions have been used where required for advanced analysis. The factors of safety presented in this manual are based on 2-D analyses.

7.9.2 Verification of Slope Stability Analyses. The following statement from EM 1110-2-1902 is applicable to levee design:

> Verification of the results of stability analyses by independent means is essential. Analyses should be performed using more than one method, or more than one computer program, in a manner that involves independent processing of the required information and data insofar as practical, to verify as many aspects of the analysis as possible. Many slope stability analyses are performed using computer programs. Selection and verification of suitable software for slope stability analysis is of prime importance. It is essential that the software used for analysis be tested and verified, and the verification process should be described in the applicable design and analysis memoranda (geotechnical report). Thorough verification of computer programs can be achieved by analyzing benchmark slope stability problems. Benchmark problems are discussed by Edris et al. (1992) and Edris and Wright (1992).

7.9.3 While software developers have performed extensive testing of programs, errors may still occur and verification is needed for all projects. Not every levee section analyzed requires verification; instead, random sections should be checked.

7.9.4 More recent publications (e.g., Duncan 2013) suggest that verification and validation be performed through duplicate analyses being performed by a different analyst using different software. This suggestion recognizes that errors are more likely due to the analyst inaccurately inputting data for analysis into the software application, rather than errors in the calculations performed by the software codes themselves. This manual does not require that this suggestion of using two different software input by two different analysts for analyses be implemented on most levee projects unless deemed appropriate by the designers, however the requirements in EM 1110-2-1902 remain applicable as stated in Section 7.9.2.

7.9.5 Verification of Seepage Analyses. Designers should consider measures to verify the results of seepage analyses that impact slope stability. There is potential for large errors in engineering judgment when performing steady state seepage analyses that influences slope stability analyses. The observational method and levee seepage performance observations described in Chapter 6 are critical for calibration of seepage analyses.

7.10 Best Practices for Slope Stability Methods.

7.10.1 Slip Surfaces (circular, noncircular, optimization).

7.10.1.1 Modern limit equilibrium method (LEM) based computer programs available for analyzing slope stability require the 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 and the associated factor of safety for this surface (and, de facto, for all other non-critical surfaces) must meet specified criteria. Most programs have search algorithms used to find the most critical slip surface, but the responsibility of locating the most critical slip surface lies with the analyst. Duncan et al. (2014) offer the following guidelines for searching for a critical slip surface:

> (1) Start with circles. It is almost always preferable to begin searching for a critical slip surface using circles. Very robust schemes exist for searching with circles, and it is possible to examine a large number of possible locations for a slip surface with relatively little effort on the part of the user.
>
> (2) Let stratigraphy guide the search. For both circular and noncircular slip surfaces, the stratigraphy often suggests where the critical slip surface will be located. In particular, if a relatively weak zone exists, the critical slip surface is likely to pass through it. Similarly, if the weak zone is relatively thin and linear, the slip surface may follow the weak layer and is more likely to be noncircular than circular.
>
> (3) Try multiple starting locations. Almost all automatic searches begin with a slip surface that the user specifies in some way. Multiple starting locations should be tried to determine if one location leads to a lower factor of safety than another.
>
> (4) Be aware of multiple minima. Many search schemes are essentially optimization schemes that seek to find a single slip surface with the lowest factor of safety. However, there may be more than one “local” minimum and the search scheme may not necessarily find the local minimum that produces the lowest factor of safety overall. This is one of the reasons why it is important to use multiple starting locations for the search.
>
> (5) Vary the search constraints and other parameters. Most search schemes require one or more parameters that control how the search is performed. Input data should be varied to determine how these parameters affect the outcome of the search and the minimum factor of safety. For example, some of the parameters that may be specified include:

a) The incremental distance the slip surface is moved during the search

b) The maximum depth for the slip surface

c) The maximum lateral extent of the slip surface or search

d) The minimum depth or weight of the soil mass above the slip surface

e) The maximum steepness of slip surface where it exits the slope

f) The lowest coordinate allowed for the center of the circle (e.g., to prevent inversion of the circle)

7.10.1.2 Advances in computing capacity have significantly increased the capability to model complex sections, stratigraphy, shear strength variations, loads, pore-water pressures, etc., and have increased the complexity of analytic methods, all of which has led to more refined slip surface searches and shapes. Computer programs may offer refined optimization schemes such as segmenting the critical slip surface into smaller linear parts and incrementally moving endpoints until a minimum factor of safety is found. These tools can be very helpful in refining the shape of the critical slip surface and often result in lower factors of safety. In some situations, the optimized slip surface produces shapes that are not consistent with the limit equilibrium method and resulting factors of safety can vary significantly. When the optimized slip surface factor of safety differs by more than 0.1 from that of the non-optimized critical circular or non-circular analyses, the analyst should determine whether the optimization result is valid.

7.10.2 Surface Slides. Experience indicates that shallow slides may occur in levee slopes after heavy rainfall. These slides are often called "slough slides." Failure generally occurs in highly plastic clay slopes. They are probably the result of shrinkage and cracking during dry weather and moisture gain during wet weather with a resulting loss in shear strength due to a net increase in water content, plus additional driving force from water in cracks. Repair of these shallow slides is normally considered to be a maintenance issue, and these failures could be eliminated or reduced in frequency by flattening the slopes, using less plastic soils near the surface of the slopes, adding drainage, incorporating geosynthetic reinforcement, or by chemical stabilization of the surface soils. Use of a curved failure envelope, as compared to using a constant friction angle with no cohesion, when defining drained strength envelopes will often help avoid search results that converge to these often trivial and less important solutions. While minimum slide depth or minimum slide weight approaches may be used to limit slip surface searches it is preferred to use a curved drained strength envelope.

7.10.3 Tension Cracks. Tensile forces from cohesion are discussed in Appendix C of EM 1110-2-1902, and the recommendations in EM 1110-2-1902 for accommodating tension cracks in LEM slope stability analyses should be followed for levee evaluation and design. Numerical instability can be a problem when using cohesion in low stress regions. An alternative to using tension cracks is to specify zero shear strength in the negative normal stress range in conjunction with using general, non-linear data point entry method (normal stress and shear stress data points are specified to represent the shear strength envelope, linear or curvilinear). Curved shear strength envelopes are often used on compacted clay embankments to represent strength from pseudo over-consolidation due to compaction. When performing slope stability using the FEM, it is common to set tension to zero. As discovered during Hurricane Katrina, tension cracks between embankment soils and structural elements can develop and fill with water, increasing driving load and reducing effective stresses; analysts should run analyses with and without a water filled “gap” to assess the potential impact on analysis results.

7.10.4 Lateral Variation in Shear Strength. Cohesion may vary with depth and with lateral location within a model. Such variation may exist due to the presence of an existing levee or other feature that creates differing pre-consolidation stresses within a soil stratum. In the example of an existing levee, the foundation strength at the centerline of the existing levee may significantly differ from that at the levee toe. Slope stability analyses need to accommodate this variation either through shear strength options within the slope stability software or by creating separate soil units for analysis, laterally within the same strata, that experience significant differences in consolidation stresses and associated strengths.

7.11 Final Design and Evaluation.

7.11.1 As discussed in Chapter 1, there are two steps required for the design of levees. The first step is for a deterministic evaluation and design to be completed using the criteria presented in this chapter. The deterministic evaluation and design are only the starting point for the levee evaluation and design. Experience has demonstrated that strictly complying with deterministic slope design criteria does not always produce levees that have the expected level of reliability. It has also been observed that blind compliance with deterministic criteria sometimes results in levees that are “overdesigned” with features that are not actually improving levee performance and reducing levee risk. The second required step in design is to evaluate and adjust the initial design using a risk assessment. The risk assessment will also serve the basis for deciding to upscale the design to be more robust than required by the deterministic criteria or to be downscaled to allow for use of reduced design criteria. A formal design deviation must be submitted in compliance with applicable USACE policy for approval before deterministic criteria and factors of safety lower will be considered acceptable.

7.11.2 The purpose of the final evaluation and design is to ensure the goals (i.e., flood risk reduction, costs, environmental benefits, etc) of the levee project are achieved. During the final evaluation and design, the levee project will be assessed for stability potential failure modes and the risk (e.g., hazard, performance, consequences) associated these failure modes are estimated. For levee slope design, identifying and evaluating stability potential failure modes is required as part of final evaluation and design. Guidance on evaluating stability potential failure modes is provided within the subsequent paragraphs in this section. It is important that stability potential failure modes for a levee project are evaluated for a range of flood loading conditions including the loading cases listed in Section 7.7. The duration of the flood event should also be considered when evaluating performing the final evaluation and design. Refer to Chapter 1 on the process that should be followed for final evaluation and design.

7.11.3 Stability Potential Failure Modes Evaluation. Stability potential failure modes occur from lowering the levee crest due to shear failure of the levee embankment and foundation or instability of the levee slopes leading to overtopping, erosion, and breach. For construction of new levees, it may be due to soft foundation conditions and overly steep slopes. For completed and existing levees, it is often caused by high pore pressures associated with seepage. Shallow slides (or maintenance slides) are generally not significant enough to cause a levee breach but could weaken the levee and make it more susceptible to other modes of failure, especially seepage. Stability potential failure modes are 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 vary over time
- Weight of the levee embankment and foundation
- Geometry of the levee and adjacent ground surface, which may vary over time, especially in areas vulnerable to erosion or change in land use

7.11.3.1 When evaluating levee stability potential failure modes, the loading conditions that should be considered include end-of-construction (including staged construction on soft foundations), floods, sudden drawdown, and seismic. A more detailed summary of loading conditions for levee and foundation stability evaluations for the end-of-construction, flood, and sudden drawdown loading cases is provided in Section 7.11.2. The requirements for evaluation of the seismic stability condition for levees are outlined in Section 7.11.4

7.11.4 <u>Seismically-Initiated Stability Potential Failure Modes Evaluation.</u>

7.11.4.1 Seismic events can trigger instability (e.g., lateral spreading) and overtopping potential failure modes and internal erosion potential failure modes (e.g., concentrated leak erosion through cracking or internal migration of the embankment into open defects). Generally, for seismic-initiated potential failure modes, consideration should be given to the combined likelihood of a damaging seismic event with a coincident hydraulic loading, or a subsequent flood event soon after an earthquake before post-seismic repairs can be completed. When the likelihood of seismic damage and post-seismic flooding are high, seismic-resistant design and construction components may be appropriate. Seismic-initiated potential failure modes are affected by the following:

- Likelihood and magnitude of the seismic event
- Likelihood and magnitude of coincident hydraulic loading
- Geometry of the levee and foundation
- Presence of liquefiable soils or weak soils vulnerable to seismic softening in the levee embankment and foundation
- Presence or absence of additional redundancy and resiliency (e.g., flatter slopes, stability berms, embankment filters, conduit filters, ground improvement, shorter monolith widths, additional reinforcement, etc.)
- Likelihood that post-seismic repairs cannot be completed prior to a flood event
- Likelihood and magnitude of post-seismic flood events that exceed the compromised crest elevation or post-seismic restored crest elevation

7.11.4.2 Historically, most levees have not been designed or evaluated for seismic loading. However, levees have experienced damage during earthquakes such as the 1989 Loma Prieta earthquake in California and the 2011 Tohuku Earthquake in Japan. Levee seismic evaluation and design alterations considering the factors listed in Section 7.11.4 should be performed. This manual does not prescribe the technical engineering procedures and tools for completing a seismic evaluation for levees. The Best Practices in Dam and Levee Risk Analysis manual (USBR and USACE 2019) provides further details and discussion on assessment of seismic-initiated potential failure modes. Designers will use the most current USACE guidance, industry standards, and best practices for seismic hazard, liquefaction, and deformation evaluation. Ultimately, seismic-initiated potential failure modes must be evaluated during design with a project-specific risk assessment, and the results of the risk assessment will be used to determine if they are significant risk drivers and as the basis for making final structural alterations to levees to mitigate seismic-initiated potential failure modes.

7.11.4.3 Coincident Hydraulic Loading. The coincident hydraulic loading is defined in ER 1110-2-1806.

7.11.4.4 Seismic Vulnerability Evaluation. For all levees and floodwalls that exceed the minimum thresholds outlined in Section 7.11.4.4.1, a seismic vulnerability evaluation will be performed to estimate the scale and location of areas of potential seismic damage that need to be addressed in a post-earthquake inspection and remediation plan.

7.11.4.4.1 Minimum thresholds for performing the evaluation are as follows:

- Peak horizontal ground acceleration greater than 0.1g at free-field conditions for an earthquake with an annual exceedance probability (AEP) equivalent to the AEP of the Operating Basis Earthquake (OBE) from ER 1110-2-1806.

- Peak horizontal ground acceleration greater than 0.05g with very loose to loose foundation conditions (i.e., $N_{1,60cs} \leq 10$ bpf (10 blows per 0.3 m) per Standard Penetration Test (SPT) or $q_{c1N} \leq 60$ kPa (1253.2 psf) per Cone Penetration Test (CPT)) and for taller levees (i.e., in excess of 4.5 m (15 feet)). $N_{1,60cs}$ is the equivalent clean sand (corrected) blow count, and q_{c1N} is the normalized penetration resistance for silty sands corrected to an equivalent clean sand value.

7.11.4.4.2 Earthquake Return Periods. For the evaluation include the following earthquakes:

- Earthquake with the same AEP as overtopping.

- Earthquake with the same AEP as the Operating Basis Earthquake (OBE) from ER 1110-2-1806.

- Range of earthquakes with AEP significantly less than the OBE up to breach (based on thresholds from seismic evaluations or other information) or the Maximum Design Earthquake (MDE) from ER 1110-2-1806.

7.11.4.5 Frequently Loaded Levees Embankments or Floodwalls. A “frequently loaded” levee or floodwall is defined as experiencing a water surface elevation of 1 foot or higher above the elevation of the landside levee or floodwall toe at least once a day for more than 36 days per year on average (10 percent of the number of days in a year) (CA DWR 2012).

7.11.4.5.1 “Frequently loaded” levees and floodwalls will initially be designed for performance requirements and seismic loading from ER 1110-2-1806 to maintain the integrity of the levee or floodwall and its internal structures without significant deformation or vulnerability to internal erosion (e.g., concentrated leak erosion in cracks or internal migration of the embankment into open defects).

7.11.4.5.2 The minimum deformation criteria outlined in Section 7.7.2 of the Urban Levee Design Criteria (CA DWR 2012) will be applied to "frequently loaded" levee design that limits total deformation to 3 feet and vertical settlement to 1 foot for levees when there is a hydraulic freeboard of 5 feet or less above the levee design water surface elevation. This criterion would be applied to the seismic loading for "frequently loaded" levees in ER 1110-2-1806. Levees with rigid penetrations or appurtenances may require smaller allowable seismic deformations. Internal damage to the levee embankment (such as shear offset, crack depth, cutoff wall damage, etc.) that may reduce effectiveness to provide flood protection after an earthquake should be considered in the evaluation. Additional reliability, approaching that expected of dams, will be provided for the levee system to continue to function during and after ground motions based on the design risk assessment.

7.11.4.6 Intermittently Loaded Levee Embankments or Floodwalls. An "intermittently loaded" levee or floodwall does not meet the definition of a "frequently loaded" levee or floodwall in Section 7.11.4.5. Seismic-initiated potential failure modes can generally be excluded (i.e., non-risk drivers) for levees or floodwalls that are intermittently loaded due to the lack of water above the landside toe for a significant period of time, and if damage were to occur, interim repairs could be performed to restore some intermediate level of protection. If interim repairs cannot be completed prior to a subsequent flood or the subsequent flood is forecasted to exceed the compromised or intermediate level of protection, the leveed area would likely be evacuated.

7.11.4.6.1 The initial levee deterministic levee design does not require remedial measures to limit liquefaction or deformations. However, cost-effective seismic mitigation measures should be implemented when justified based on the results of the design risk assessment completed after initial deterministic levee design.

7.11.4.6.2 The seismic vulnerability evaluation is required when seismic-initiated potential failure modes are not considered risk drivers but the thresholds in Section 7.11.4.4.1 are met. The scope of the evaluation is to inform the post-earthquake inspection and remediation plan.

7.11.4.7 Post-Earthquake Inspection and Remediation Plan. A post-earthquake inspection and remediation plan will be completed for levees that require a seismic vulnerability evaluation. Best practices for development of a plan are found in Section 7.7.1 of the Urban Levee Design Criteria (CA DWR 2012). Emergency Action Plan (EAP) for levees with potential for seismic damage should be developed identifying measures, material sources, and other emergency response tasks to repair a levee prior to a subsequent flood.

7.12 Measures to Increase Stability.

7.12.1 Improving levee slope stability can be accomplished either by reducing the destabilizing loads or improving the resisting forces. Reducing levee loading with lightweight fill (such as geofoam or lightweight aggregates) can create robustness and resiliency concerns for hydraulic structures and is not recommended. Means for improving weak and compressible foundations can involve consolidation of foundation materials to improve soil shear strength; these design considerations are discussed in Chapter 8. Common methods for improving embankment stability include the following options:

7.12.2 <u>Flatten Embankment Slopes</u>. Flattening embankment slopes will usually increase the stability of an embankment, especially against shallow failures that takes place entirely within the embankment. Flattening slopes also spreads the embankment load more uniformly and increases the length of potential slip surfaces and thereby increase resistance to sliding, especially for deeper failure surfaces.

7.12.3 <u>Stability Berms</u>. While seepage berms add weight to the ground surface to resist upward seepage forces at the landside levee toe, sometimes they are insufficient to solely achieve the desired stability factor of safety. In those cases where a seepage berm is used, either the thickness of the seepage berm can be increased, or a stability berm can be constructed upon the seepage berm adjacent to the levee slope. Stability berms sometimes include chimney/blanket filter drains to control seepage and further enhance stability. Stability berms essentially provide the same effect as flattening embankment slopes but are generally more effective because of concentrating additional weight where it is needed most and by forcing a substantial increase in the length of the potential slip surface. Thus, berms can be an effective means of stabilization not only for shallow foundation and embankment failures but for more deep-seated foundation failures as well. Berm thickness and width are determined from stability analyses and berm length should be great enough to encompass the entire problem area, the extent of which is determined from the soil profile in conjunction with seepage and stability analyses. Some FEM seepage programs artificially lower the phreatic surface within an embankment when an external chimney drain is included in the cross-section. Analysts are cautioned to verify this condition is not occurring when performing stability analyses of an embankment with an external chimney drain/stability berm. Foundation failures are normally preceded by lateral displacement of material beneath the embankment toe and by noticeable heave of material just beyond the toe. When such a condition is noticed, berms are often used as an emergency measure to stabilize the embankment and prevent further movement.

7.12.4 <u>Internal Embankment Drainage</u>. Internal embankment drainage can significantly improve the landside levee stability under steady seepage conditions, by intercepting and lowering the phreatic surface that develops within the embankment. Incorporating drainage may be a cost-effective solution when real estate is not available to flatten levee slopes or add a stability berm.

7.12.5 <u>Foundation Improvement</u>. Foundation shear strengths can be improved by reducing pore-water pressures, creating stronger foundation materials through the introduction of secondary materials, or by pre-loading to cause settlement with subsequent reduction in void ratio and increase in shear strength.

7.12.5.1 Foundation Drainage. By reducing pore-water pressures through shallow (drain trench) or deep (relief well) drainage schemes, effective stresses will increase leading to increased shear resistance in soils characterized with frictional shear strength. Using drainage to improve stability may not be cost effective and is often only reasonable when the designer is confident that drainage will always be in place, will always be efficient, and will always be maintained. Clogging and bio-fouling often reduce drain efficiency and require unanticipated maintenance. Drain efficiency must be conservatively modeled when used on USACE projects.

7.12.5.2 Flatten Excavation and Existing Foundation Slopes. Flattening foundation slopes will usually increase stability. In many agricultural areas with reclaimed wetlands, drainage ditches to remove water from fields were often used as the borrow source for levee materials and were often excavated into native materials at the levee landside toe. These ditches exacerbate seepage problems during floods and are a common cause of slope instability. Designers and planners should avoid and if possible, backfill them with materials that provide sufficient weight to counteract uplift forces and with appropriate filtration and drainage characteristics.

7.12.5.3 Seepage Cutoff Trenches/Walls. Installation of a low hydraulic conductivity trench/wall, that cuts off seepage through aquifers extending beneath levees, reduces pore-water pressure landward of the levee. Lower pore-water pressures lead to higher effective stresses, leading to increased shear resistance in soils characterized with drained, effective stress frictional shear strength. The depth of penetration of the cutoff wall through the aquifer will control the effectiveness of this measure in reducing pore pressure.

7.12.5.4 Soil Cementation Techniques. Soil can be mixed with cement to form columns of improved materials. Two common techniques are the deep mixing method (DMM) and jet grouting. DMM involves the injection of dry or wet cement and mixing the cement with foundation materials using paddles or a cutter to create soil/cement columns. Columns may overlap to create high strength panels perpendicular to the levee centerline. Depth and spacing is varied to meet slope stability criteria. The desired end product of jet grouting is similar to DMM, however, high velocity fluid jets are used rather than mixing techniques.

7.12.5.5 Wick Drains. Wick drains, also called prefabricated vertical drains (PVDs), are used to speed consolidation in fine-grained soils. The improvement in slope stability is due to increases in undrained shear strength rather than reinforcing effects of the PVD elements. It is noted that the use of wick drains in levee foundations needs to be evaluated as to whether they could provide a preferred underseepage path and contribute to potential internal erosion, as discussed in Section 8.6.3.3. See Chapter 8 for additional information on wick drain design, and Appendix H for additional information on construction options for staged construction.

7.12.5.6 Remove and Replace. In areas where shallow soft deposits exist or failure surfaces have developed, it may be possible to excavate and remove part or all the unsatisfactory material and replace it with an engineered fill with higher shear strength properties. Rather than improving the soils in-place, such as with DMM, shallow slides on levees constructed with high plasticity clays have been repaired by excavating the slide materials, mixing with an additive such as lime, and replacing the treated material and compacting. Other techniques may be considered (such as sand trenches and aggregate piers/stone columns) to replace a percentage of the soft soil with higher strength soil. Any alternative that introduces foreign elements into either the embankment or foundation needs to be carefully considered so as not to augment another failure mode, especially seepage.

7.12.5.7 Structural Elements. There are several types and configurations of structural elements used to improve slope stability. The introduction of structural elements to improve levee stability is often more costly than earthwork solutions. However, real estate or environmental impacts, as well as economics, can result in the need to minimize the levee footprint. Soil reinforcement has been used in conjunction with mechanically stabilized walls and slopes to reduce the levee footprint in tight alignment areas. In some cases, the level of protection has been raised by constructing I-walls or T-walls atop levees, thereby not increasing the levee footprint. This composite levee/wall system must address wall modes of failure in addition to global stability. See EM 1110-2-2502 for more on flood and retaining walls.

7.12.5.8 Geosynthetic Reinforced Foundations. The introduction of high strength geosynthetics placed on or near the embankment-foundation contact has provided options for base reinforcement of levees on very soft soils. Levee stability for both long term design load cases and short term construction load cases can be increased through the use of geosynthetic reinforcement. If reinforcement is used, levee slopes may be steepened, and in that way, the levee footprint and loading are reduced. Designs involving earth reinforcement are often used in areas of limited real estate, where levee stability requires large earthwork quantities, and where the foundation soils require staged loading. Appendix H provides guidance for geosynthetic-reinforced embankments over soft soils.

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CHAPTER 8

Settlement

Section I
Introduction

8.1 General.

8.1.1 Settlement Evaluation and Monitoring.

8.1.1.1 Evaluation of the amount of post-construction settlement that can occur from consolidation of both the levee embankment and the foundation may be important if the settlement would result in a top of levee elevation below the required minimum final levee grade or damage to structures in the embankment. In addition to this manual, EM 1110-2-1904 should be used as supplemental guidance for performing settlement analyses. Detailed settlement analyses should be made when significant consolidation is expected as under high embankment loads, embankments of highly compressible soil, embankments on compressible foundations, and beneath steel and concrete structures in levee systems founded on compressible soils. Long-term settlement may also be induced by consolidation settlement and secondary compression, changes in loading conditions due to changes in land use (such as water or oil extraction), changes in hydrogeologic conditions, regional subsidence (such as in consolidating deltaic environments at the mouth of large rivers, e.g., Mississippi River in Louisiana), and sea level changes. Depending on project objectives, mitigation of these long-term sources of settlement may also be incorporated in the levee design and maintenance obligations.

8.1.1.2 Historically, “freeboard” (that is, height of the levee in excess of the design water surface elevation) was used to account for hydraulic, geotechnical, construction, operations, and maintenance uncertainties. The risk-based analysis now used to establish a nominal top of barrier accounts for hydraulic uncertainties but does not incorporate uncertainty associated with overbuild of levee height for geotechnical performance aspects. Generally, deterministic analysis using physical properties of the foundation and embankment materials will be used to estimate the required overbuild for setting the constructed top of levee to account for settlement, shrinkage, cracking, geologic subsidence, and construction tolerances. In some situations, the analysis can be expanded using probabilistic methods to explicitly assess and incorporate geotechnical settlement uncertainty.

8.1.2 Incorporating Settlement Expectations in Design and Maintenance.

8.1.2.1 Post-construction long term settlement can be accommodated by a number of means, including overbuilding the levee, constructing a floodwall, and/or raising the levee in future years with additional soil lifts. In some locales, overbuilding has been incorporated based on a certain percentage of levee height as supported by previous experience in levee projects. However, overbuild based on estimated settlement in accordance with the state-of-practice procedures for settlement calculation are generally considered more appropriate. Overbuilding may increase the severity of stability problems and may be impracticable or undesirable for some foundation conditions. Partially overbuilt levees with plans for future lifts, allowing consolidation and associated strength gains to offset the impact of destabilizing weight, can also be considered.

8.1.2.2 When making modifications to existing levees, predicting further settlement can be difficult. Existing levees may have a history of raises or repairs at different times and of different magnitudes that complicates estimating effective stress changes, corresponding volume changes, and degree of consolidation profiles. When existing levees are raised, it is necessary to predict the remaining settlement that may be a combination of residual settlement of the existing section and additional settlement due to the placement of new fill. The estimate of final settlement is then considered in the needed overbuild for the levee project or considered in the evaluation and final levee height for existing levee evaluations.

8.2 Factors Affecting Levee Settlement.

8.2.1 Long term settlement may have several components. Consolidation and secondary compression settlement may occur due to change in loading and pore pressure conditions. Fine-grained soils usually exhibit higher long term settlement which may vary based on soil properties.

8.2.2 Compaction procedures during embankment construction may also affect settlement. For example, post-construction settlement, such as hydrocompaction, may occur if soil is compacted in a dry state and at a low density. In such cases, the embankment may exhibit settlement when moisture is added by rain or high water events.

8.2.3 Differential settlement may occur due to differing geologic conditions such as the presence of soft organic soils or historic channels. When settlement differences occur over short distances, the differential movement can result in shear or cracking that could contribute to seepage related problems, and it may be prudent to perform excavations along these areas to slope back and reduce abrupt changes in foundation topography. Pockets of soft sediments and incorporation of hard elements (such as pile founded walls or cement-bentonite slurry walls) within embankments that have not fully consolidated can lead to differential settlement that must be anticipated and understood for predicting, and possibly mitigating for, unacceptable levee performance.

8.2.4 Local or regional subsidence may increase settlement potential of existing levees. Local subsidence may occur due to oxidation of surficial organic layers or change in localized groundwater use. Regional subsidence may occur due to change in regional groundwater conditions. Subsidence may generally affect the entire levee, or it could be differential based on foundation conditions.

Section II
Settlement Computation

8.3 General.

8.3.1 Settlement estimates can be performed using the state-of-practice procedures commonly used for embankment loading. The following sections are brief summaries of the generalized settlement computation methodology. EM 1110-2-1904 provides further guidance for settlement estimates. Detailed settlement analyses should be made when significant consolidation is expected, as under high embankment loads, embankments of highly compressible soil, embankments on compressible foundations, and beneath steel and concrete structures in levee systems founded on compressible soils.

8.3.2 The total settlement, S_t, of a loaded soil has three components as shown in Equation 8-1.

$$S_t = S_i + S_c + S_s \tag{8-1}$$

where:
S_t = the total settlement
S_i = the immediate settlement
S_c = the consolidation (time-dependent) settlement
S_s = the secondary compression (time-dependent) settlement

8.3.3 Most settlement that occurs in coarse-grained soils is immediate. The consolidation settlement is a time-dependent process that occurs in saturated fine-grained soils that have low hydraulic conductivity. The rate of settlement in such cases depends on the rate of pore water drainage. Secondary compression occurs at constant effective stress and with no subsequent changes in pore water pressure (Holtz et al. 2011).

8.3.4 The actual service life of a levee typically exceeds the economic life used to calculate life cycle costs. Settlement then must be computed, and overbuild provided, to accommodate 100 percent of the expected consolidation throughout the operating life. Levees do not need to be designed for 100 percent of the expected consolidation if there is an authorized plan for future lifts/raises to maintain the design height over the life of the project. Settlement calculations are typically based on data from consolidation testing. The coefficient of compression for design is selected for the soil strata and levee reach considered and the resulting settlement is used as the basis for selecting the levee overbuild for that reach. Settlement is computed for each design reach and the overbuild height is added to the final levee grade along the project in a way that minimizes the number of changes in grade. For example, if a 6-inch overbuild is required in one reach while a 4-inch overbuild is needed in an adjacent reach, the 6-inch overbuild could be applied to both reaches. Overbuild for settlement is always applied in increments such that the settled embankment is at or above the final levee grade established by hydraulic requirements.

8.4 Settlement Computation.

8.4.1 General Steps in Settlement Analysis. The general steps for settlement analyses of levees are summarized below.

8.4.1.1 Step 1: Determine the initial conditions. Within the soil profile, estimate the existing vertical total stress (σ_{v0}), the existing pore water pressure (u_0), and the existing effective stress (σ'_{v0}). Estimate the soil properties including the preconsolidation pressure (σ'_p), compression index or modified compression index (C_c or $C_{c\varepsilon}$), recompression index or modified recompression index (Cr or $C_{r\varepsilon}$), coefficient of consolidation (c_v), and secondary compression index or modified secondary compression index (C_α and $C_{\alpha\varepsilon}$). The soil profile and soil properties should be developed based on subsurface investigations (Chapter 2) and laboratory testing (Chapter 3). The soil properties derived from laboratory tests should be compared with published values or local practice for normally to slightly consolidated soils with similar water contents and Atterberg limits to evaluate the appropriateness of the laboratory test results.

8.4.1.2 Step 2: Determine the geometry and magnitude of loads on the foundation to be used in the design. Levee dimensions are developed from project drawings.

8.4.1.3 Step 3. Estimate the change in stress on the compressible layer. The change in stress conditions can be developed using procedures outlined in EM 1110-2-1904. Finite element models can also be utilized to determine the stress distribution with additional loading due to levee construction.

8.4.1.4 Step 4: Estimate the preconsolidation pressure. Based on comparison of preconsolidation pressure (σ'_p), existing effective vertical stress (σ'_{v0}), and overconsolidation ratio (OCR), evaluate whether the soil layers are normally to slightly over-consolidated or over-consolidated. Soil layers may include both normally to slightly over-consolidated and over-consolidated layers.

8.4.1.5 Step 5: Estimate the immediate settlement of fine-grained soils using elastic theory and estimate immediate settlement of coarse-grained soils using empirical methods as described in EM 1110-2-1904.

8.4.1.6 Step 6: Estimate the consolidation settlement based on soil properties, load, and OCR as described in EM 1110-2-1904.

8.4.1.7 Step 7: Estimate the rate and magnitude of secondary compression. Based on boundary and drainage conditions, establish the parameters for secondary compression such as the excess pore water pressure at the beginning of consolidation and rate of dissipation. Estimate the rate and magnitude of secondary compression and evaluate how to address the consolidation and secondary compressions.

8.4.2 Estimating Remaining Settlement. Design computations along with as-built information and current surveys can be used to estimate remaining settlement for conditions where the embankment section and foundation conditions remain consistent with the original design. An estimate of remaining settlement can be assessed where frequent and reliable survey data is available to develop time/settlement curves. However, there may be situations where variable loading history, uncertain benchmarks used in construction, and conflicting data diminish the confidence in predicting future settlement. In those situations, piezometers may be used to assess pore-water pressure conditions that exist in excess of hydrostatic. The change in pore-water pressure then denotes the change in effective stress expected and corresponding volume changes can be estimated. Installing piezometers (e.g., fully grouted vibrating wire transducers) along with automated data acquisition systems or other means of collecting data are discussed in EM 1110-2-1908.

8.5 Instrumentation and Monitoring. Monitoring of levee crown elevations can provide useful information regarding long term settlement conditions. If possible, settlement monitoring instruments should be installed in existing and new levees for long term settlement monitoring. Surveying of these instruments can be useful in comparing top of levee locations (such as along crown centerline and hinge points) with design. Where applicable, an evaluation of regional subsidence trends should be performed. EM 1110-2-1908 provides guidance on deformation monitoring instrumentation. Air-borne or ground-based light detection and ranging (LIDAR) can be used to develop topography which can also be used to evaluate settlement. As part of the National Levee Safety Program, information on levee profile elevations is now routinely stored in the National Levee Database (2007).

Section III
Mitigating for Settlement

8.6 Soil Improvement Methods. Levees are often constructed over areas with highly variable subsurface conditions. While it is desirable to construct levees in foundation conditions that would require minimum post-construction measures to account for settlement due to alignment constraints, it often becomes necessary to construct levees across highly compressible foundations. EM 1110-1-1904 provides additional information on improvement methodologies and discusses applications that cover a wide range of soil types; however, many of the alternatives presented apply to non-levee projects or are not routine for levee construction. The following paragraphs expand on the procedures that are commonly applied to levee projects.

8.6.1 Remove and Replace. Removal of soft soils in the levee foundation and replacement with satisfactory fill compacted in lifts may be used to reduce settlement in areas where shallow soft deposits or fill layers exist. This alternative becomes less feasible where compressible layers are deep or where high water tables exist that would require dewatering during construction.

8.6.2 Staged Construction. Excessive levee settlement typically coincides with low shear-strength soils and slope-stability problems. In order to minimize the levee footprint , while still meeting stability criteria, the levee may be constructed in stages. Staged construction requires adequate time for consolidation and subsequent strength gain in the foundation, before adding another levee lift. The designer is involved with taking and analyzing the settlement monitoring instrumentation readings during construction in order to assess when the next levee lift can be placed. Instrumentation for monitoring staged construction often includes settlement gages placed at the interface between the levee and foundation, surface monitoring points for surveys following fill placement, and piezometers to track pore-water pressure dissipation. In some cases, levee lifts can be scheduled to be placed after completion of the original construction. In such cases, a monitoring program should be implemented to compare the settlement progression with design estimates and timing of later levee lift placements may be adjusted if needed. Staged construction is typically accompanied by strength gain verification program prior to constructing additional lifts that includes field and laboratory testing. There may be projects where future levee lifts will be required to maintain levee crest height. For federally constructed projects, construction of those lifts may be the responsibility of the non-Federal sponsor. An example lift schedule for such a project is shown in Figure 8-1. This lift schedule presents the expected year of levee raise along with the expected height. In this example the design levee grade increases with time due to consolidation, projected sea level rise, and regional subsidence. Designers should account for uncertainty in settlement estimates (i.e., both in magnitude and time rate of consolidation) when developing and planning lift schedules

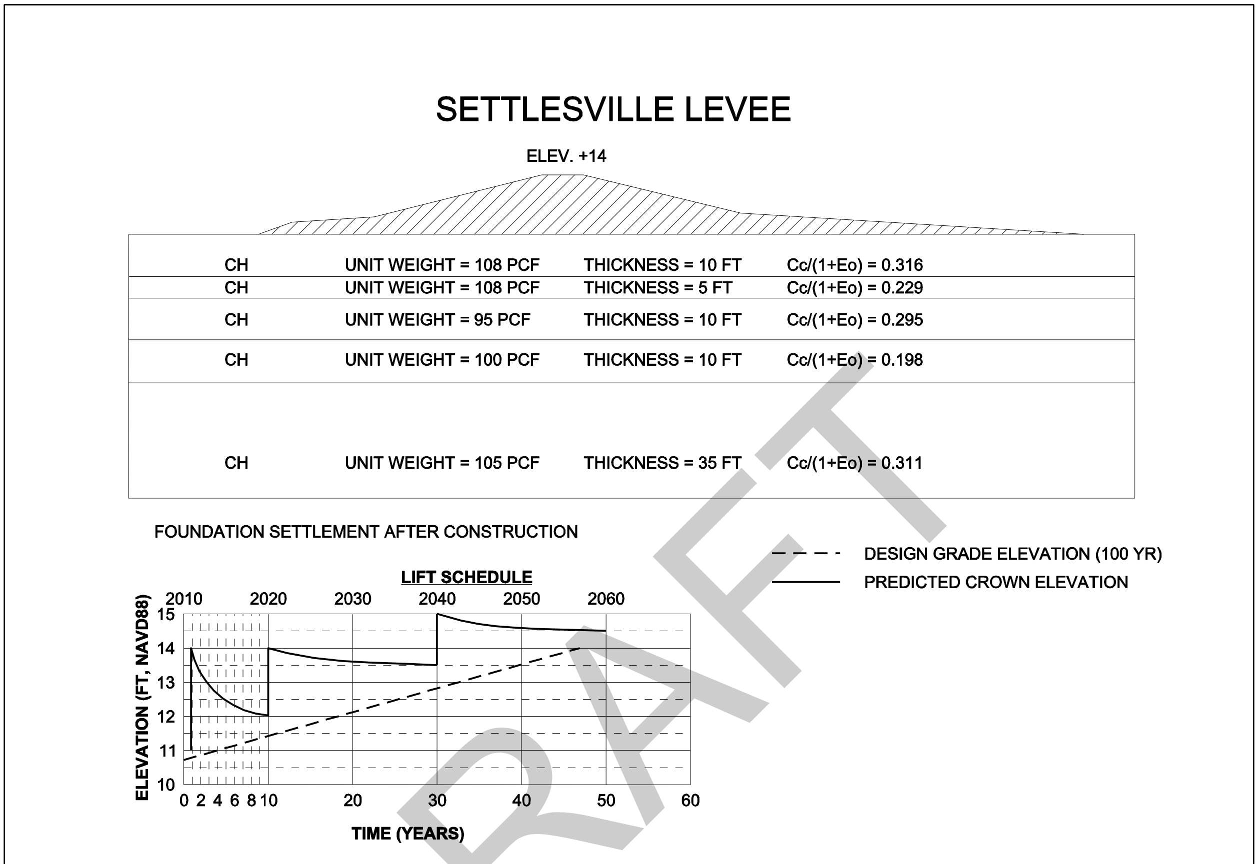

Figure 8-1. Settlement Profile and Example Lift Schedule showing Years and Expected Magnitude of Levee Raises.

8.6.3 Prefabricated Vertical (Wick) Drains.

8.6.3.1 Staged levee construction over very soft foundation soils with low hydraulic conductivity have been expedited with the use of wick drains. The design is optimized by maximizing the wick drain spacing to achieve an appropriate degree of consolidation needed within the time available for the consolidation. Usually the project schedule controls the available time for construction so the design must accommodate a satisfactory strength gain of consolidating layers, associated with the average degree of consolidation of the foundation materials, so the levee lifts can be safely placed to the required levee grade.

8.6.3.2 Wick drain design can be performed in accordance with FHWA (1986) with considerations for effects of wick drains in levee performance during high water events.

8.6.3.3 The long term performance of these wick drains should be considered in seepage evaluations also. If a drainage system is properly designed for collection of excess pore-water during high water conditions, these wick drains have potential to act as a pressure relief system. However, such drainage systems should not connect the waterside of a levee to the landside, as this may result in excessive seepage flow and an unsafe condition during high water events. In absence of a properly designed drainage system to collect excess pore-water, these wick drains may shorten the seepage path and may negatively impact blanket layers. The reduced thickness of any fine-grained blanket should be considered in long-term levee seepage evaluation.

8.6.4 Preloading and Surcharge Fills. Preloading involves placement of fill preceding levee construction to compress the foundation materials prior to placing levee fill. Preloading and surcharging can be extremely valuable at structure locations or at levee-structure transitions to minimize foundation consolidation after the structures are constructed. Preload material is typically material not meeting levee fill requirements and is placed before levee construction and removed before final levee construction. Preloading using levee materials that remain in place is termed staged construction. Where stability conditions allow, surcharge placed to heights in excess of the final levee height (that is, fill exceeding that required to achieve a given settlement) may be placed to accelerate the consolidation time needed during construction or achieve the required settlement anticipated by levee loading.

8.6.5 Soil Improvement/Amendment. Levee foundation materials can be improved in-place or amended and replaced following excavation. As discussed in Section 8.6.1and in EM 1110-1-1904, soft foundations can be excavated; rather than replacing with a borrow material, the excavated material may be treated (such as drying) and replaced in lifts and compacted. If any additive is used in soil improvement, the effects on hydraulic conductivity and strength should be evaluated and measures taken to avoid any negative impact. Deep mixing methods (DMM) are in-situ techniques that have been used to reduce levee settlement, often in conjunction with improving levee stability. However, DMM introduces hardened elements in the levee and/or levee foundation which can cause differential settlement. Projects that are candidates for use of DMM are those where levee materials are compliant and can deform without cracking rather than those that are stiff or hard.

8.6.6 Pipes. Differential settlement between the levee centerline and the levee toe may require gravity drains or conduits to be cambered when installed (that is, the pipeline under the levee centerline is built higher than the pipeline at the levee toes). When the settlement occurs, the pipeline should not have a sag that will puddle water and promote sediment deposition. Guidance for pipes, culverts and other conduits through levees can be found in EM 1110-2-2902.

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CHAPTER 9

Erosion Evaluation and Design

9.1 General.

9.1.1 This chapter presents methods to evaluate the need for erosion protection and describes general approaches to design and construct levees to resist surficial erosion caused by river current, wave action, and overtopping steady flow and waves. It should be noted that the state-of-the-practice is not as well developed for erosion evaluation and design at this time as it is for other aspects of levee design; analysts and designers are encouraged to seek the best available information from other reliable sources when evaluating erosion potential and designing mitigation measures.

9.1.2 Section 1.4 of Chapter 1 outlines the required loading conditions for evaluation and design of levees. This manual does not provide factors of safety like seepage and slope stability criteria for erosion; however, it does require that designers explicitly evaluate and design for surficial erosion. Levees shall be designed to minimize or eliminate damage to the levee for flood loadings up to the Design Water Surface Elevation (DWSE) . Designers are also required to complete appropriate engineering analyses to assess erosion potential failure modes to the Top of Levee (As-Constructed and Final Levee Grade) condition as outlined in Section 1.4. Similar to other failure modes, designers will use the Top of Levee analyses results to inform any design adjustments with a risk assessment (e.g., Phase 2).

9.2 Surficial Erosion Potential Failure Modes (Including Overtopping).

9.2.1 Erosion is one of the principal causes of levee damage and can lead to both overtopping and prior-to-overtopping failures. Surficial erosion potential failure modes occur when the loss of the levee crest is caused by erosion of the levee embankment or foundation. Surficial erosion potential failure modes may be caused by overtopping of the levee embankment due to static high water or intermittent wave over-wash, by waterside erosion of the levee embankment due to wave action on the waterside of the levee, or by high velocity channel flows flowing across the waterside slope. Overtopping-related potential failure modes are often the most significant risk driver for levees and are affected by the criteria used to establish the top of levee elevation (i.e., the height of the levee vis-à-vis the frequency of loading) and the erodibility of the levee soil and any armoring materials, used to prevent erosion. Waterside erosion-related potential failure modes can also be significant for levees adjacent to steep-grade streams, levees near the stream or riverbank, and levees in coastal areas.

9.2.2 Even more than other potential failure modes, erosion has an explicit dependence on both water levels and duration of loading, making analysis more complex. Evaluation of surficial erosion generally requires more interaction between geotechnical and hydrologic/hydraulic members of the project delivery team.

9.2.3 Poor levee performance due to surficial erosion is increased by a number of factors which may include compromised levee prism geometry, geomorphologic trends as indicated by historical damage (for example, river channel migration, and shoreline recession), river-flow velocity-induced shear, wind-wave shear stress, embankment and foundation erodibility, the presence of harmful vegetation, animal activity, and/or absence of erosion protection. For riverine levees, erosion will most likely be due to a weakened levee cross-section coupled with high flow velocity. In large, open bodies of water like a wide river bypass or coastal settings, wind-wave damage is expected to be a dominant cause of erosion and can affect both the waterside and landside slopes and crest. Erosion is also caused by factors such as surface runoff and boat wakes, though these forms of erosion do not frequently contribute significantly to risk and are mitigated through routine maintenance activities. Figure 9-9-1 shows the progression of riverine current erosion, flood fighting repairs, and post-flood evidence of damage at a Missouri River levee site during the record 2011 flood. Some of the most common erosion failure modes that should be considered during evaluation and design are listed below (paragraph 9.2.4).

(a) River flow eddies noted during helicopter survey 7/15/2011
(b) Arcuate vertical scarp on waterside slope 7/23/2011
(c) Emergency armoring and spur dike construction 7/27/2011
(d) Undermining of waterside slope and riverbed 11/1/2011

Figure 9-9-1. Missouri River Levee System (MRLS) 488-L during and after the 2011 Missouri River Flood (Photos by USACE Kansas City District).

9.2.4 Typical erosion potential failure modes for levees include:

(Note: this list is not exhaustive).

- River current with sufficient depths, velocity, and turbulence to erode waterside embankment and foundation materials contributing to increased seepage, slope instability, and mass wasting that progresses to breach.

- Fetch and wind on large water body next to levee generates waves large enough to erode waterside armoring and underlying embankment and foundation materials contributing to increased seepage, slope instability and mass wasting that progresses to breach.

- River or coastal overtopping of levee leading to flows high enough to erode landside armoring and underlying embankment and foundation materials contributing to landside head-cutting, increased seepage, and slope instability that progresses to breach.

- Coastal storm surge combined with wind waves large enough to run-up and overtop levee leading to turbulent transient flows high enough to erode landside armoring and underlying embankment and foundation materials contributing to landside head-cutting, increased seepage, and slope instability that progresses to breach.

- Debris or ice jams cause localized scouring or water surface elevation increases that overtop the levee.

9.3 Key Factors to Consider for Erosion Potential Failure Modes.

9.3.1 In general, erosion common for levees can be grouped into four types: waterside surficial current erosion, waterside wind-generated wave action erosion, landside overtopping flow erosion, and landside overtopping wave action erosion.

9.3.2 The key factors that impact levee erosion are fundamentally similar and include embankment materials, embankment geometry, and hydraulic loading. The difference is what is driving the potential failure mode (riverine, wind, and/or surge) and the location along the levee where the erosion is most likely to occur.

9.3.3 Waterside surficial current erosion and waterside wind-generated wave erosion can occur when the water level is below the top of the levee (that is, without overtopping). Surficial current erosion is caused by rapid flow of water, generally parallel to the waterside slope but can also occur if conditions within the floodway result in directed impinging flow onto the streambank and/or embankment. Waterside slope erosion by waves can occur from two mechanisms: by generating excess shear stress on the soil underneath the waves (bottom currents) or by wave breaking on the levee slope.

9.3.4 Overtopping flow and overtopping wave action erosion are caused by water flowing over the top of a levee when flood and/or wave heights and associated wave run-up exceed the height of the levee. Overtopping flow is characterized by flood waters rising above the levee crest and flowing over the top of the levee and can happen in both riverine and coastal settings. Overtopping wave action is characterized by a storm event causing a surge, thus raising the water on the levee, combined with wind generated waves that break near or at the crest and run up and over the crest and down the landside slope. Overtopping wave action erosion is more likely to occur in a combined event of wind and coastal storm surge flooding or along a bypass floodway with a long wind fetch.

9.3.5 The embankment materials resistance to erosion will depend on the behavior of the soil material subjected to the shear stress of the water. The erodibility of soils can be affected by various factors including soil composition, grain size distribution, compaction characteristics, and degree of cementation. Methods to evaluate and quantify erodibility of soils have been studied for years and have usually involved laboratory or field tests of soil samples. Representative examples from the literature include flume tests, jet erosion test (JET), rotating cylinder test (RCT), erosion function apparatus (EFA), and hole erosion test (HET). Different parameters such as erosion coefficients and critical shear stresses obtained from these tests have been used to estimate the erosion rate and the critical point of shear stress when erosion starts with the intent to use it for modeling erosion. Results of these tests have provided the data to develop erosion categories that can vary from very resistant to very erodible. While these tests have been used for many years to understand the characteristics of the erosion behavior of soils, it has been difficult to compare the results of one test to the other due to the different erosion mechanisms and the stress environment of the samples created by each test. Briaud et al. (2019) compiled the most comprehensive data set of soil tests conducted around the world and provides correlation equations for erodibility of soils based on the different erosion tests and soil properties. The following findings from this study relate the geotechnical properties to the erosion resistance in the different soil erosion tests.

- For soils with a mean particle size D_{50} greater than 0.3 mm, an increase in D_{50} leads to an increase in the resistance of the soils to erosion. For soils with a D_{50} less than 0.3 mm, an increase in D_{50} leads to a decrease in soil resistance.
- If D_{50} is less than 0.074 mm (fine-grained soils), a decrease in the coefficient of curvature or uniformity increases the resistance to erosion.
- Increasing clay content in fine- and coarse-grained soils increases the resistance to erosion.
- While there are some exceptions, in general, an increase in the plasticity index can increase resistance to erosion in fine- and coarse-grained soils.
- An increase in plastic limit increases resistance to erosion in fine-grained soils.
- If D_{50} is less than 0.3 mm, in many cases, the wet unit weight and undrained shear strength are proportional to the resistance to erosion.
- While water content alone does not provide good correlation with erosion resistance, it seemed to have a positive effect on finer soils in the majority of tests and a negative effect on coarse-grained soils in the EFA apparatus.

It is important to understand that most erosion tests are usually conducted on samples that are compacted and tested at the same water content, immediately after compaction, which may not reflect in-situ conditions. Furthermore, both native and engineered fill materials are subject to various processes, such as shrinking and swelling with seasonal variations in moisture; this may result in cumulative change in erosion characteristics over time, with deeper material being less and shallower material being more susceptible to those changes. Finally, the erodibility of natural soils may be affected by geologic processes that will increase or decrease the erosion resistance compared to recompacted soil samples. Using experience and engineering judgment, engineers must protect susceptible locations where the combination of soil erodibility and high shear stresses could compromise the integrity of the structure.

9.3.6 The geometry of the levees influences the erosion potential. Flatter slopes and wider crests would lead to a wider erosion failure path. A flatter slope may reduce wave runup because of the longer distance the wave will travel up the slope reducing the overtopping discharge rate. Limits on overtopping discharge rates are available in the EurOtop Manual on wave overtopping of sea defenses and related structures. The angle of the slope will also affect how the wave breaks which in turn influences the wave runup and overtopping discharge rate, EM 1110-2-1100 Part VI Chapter 5 provides additional information on wave runup and rundown on a structure.

9.3.7 Velocity is a key component of erosion potential failure modes. There is a variety of modeling software available to obtain velocities in the river channel, along riverbanks, and on levee slopes. Velocities should be considered for a variety of loading conditions since flows ranging from low flows to overtopping the levee can result in erosion potential failure modes. EM 1110-2-1601 gives values of permissible mean channel velocities that are widely used as a guide to design non-scouring grass-lined earth channels (Table 9-1) and which can be considered when designing levee waterside slope erosion protection. USDA-SCS (1984) also provides estimates of permissible velocities for channels lined with vegetation (Table 9-2).

Table 9-1. Suggested Maximum Permissible Mean Channel Velocities (Source: EM 1110-2-1601)

Channel Type	Grass	Channel Material	Mean Channel Velocity (ft/s)
Grass-lined earth channel	Bermuda Grass	Sandy Silt Silt Clay	6 8
	Kentucky Bluegrass	Sandy Silt Silt Clay	5 7

Table 9-2. Permissible Velocities for Channels Lined with Vegetation

Cover	**Slope Range**[2]	**Permissible Velocity**[1]	
		Erosion-Resistant Soils[3]	**Easily Eroded Soils**[4]
	Percent	*m/s (ft/s)*	*m/s (ft/s)*
Bermuda grass	<5 5-10 Over 10	2.43. (8) 2.13 (7) 1.82 (6)	1.82 (6) 1.22 (4) 0.91 (3)
Bahia grass Buffalo Grass Kentucky bluegrass Smooth brome Blue grama Tall fescue	<5 5-10 Over 10	2.13 (7) 1.82 (6) 1.52 (5)	1.52 (5) 1.22 (4) 0.91 (3)
Grass mixture Reed canarygrass	[2]< 5-10	1.52 (5) 1.22 (4)	1.22 (4) 0.91 (3)
Lespedeza sericea Weeping lovegrass Yellow bluestem Redtop Alfalfa Red fescue	[5]<5	1.06 (3.5)	0.76 (2.5)
Common lespedeza[5] Sudangrass[6]	[7]<5	1.06 (3.5)	0.76 (2.5)

1 Use velocities exceeding 1.52 m/s (5 ft/s) only where good covers and proper maintenance can be obtained.
2 Do not use on slopes steeper than 10 percent except for vegetated side slopes in combination with a stone, concrete, or highly resistant vegetative center section.
3 Cohesive (clayey) fine-grained soils and coarse-grained soils with cohesive fines with a plasticity index of 10 to 40 (CL, CH, SC, and CG).
4 Soils that do not meet requirements for erosion-resistant soils.
5 Do not use on slopes steeper than 5 percent except for vegetated side slopes in combination with a stone, concrete or highly resistant vegetative center section.
6 Annuals – use on mild slope or as temporary protection until permanent covers are obtained.
7 Use on slopes steeper than 5 percent is not recommended.

Source: USDA-SCS (1984), Chapter 7, “Grassed Waterways and Outlets”, page 7-14

9.3.8 Location. Erosion potential failure modes can impact the levee embankment in several manners and locations. Erosion on the riverbank may encroach on the levee section and result in erosion in the levee foundation, see Figure 9-2 below, or erosion on the bench may encroach on the levee toe. Erosion may also occur on the embankment itself; this erosion could be exacerbated by the presence of lone trees (Figure 9-3), debris, or ice jams (Figure 9-4). Riverine erosion is of particular concern in areas of river bends, due to an increase of the velocity on the outside of the bend. EM 1110-2-1601 provides guidance regarding scour in river bends, meandering channels, and braided channels.

Figure 9-2. Erosion Below Levee Toe in Foundation (USACE Sacramento District).

Figure 9-3. Erosion Above Levee in Embankment Due to Lone Tree (USACE Sacramento).

Figure 9-4. Ice Jam along levee on Platte River (USACE Omaha District).

9.3.9 Overtopping flows may result in erosion on the levee crest and/or landside of the levee and is of particular concern in areas of tie-in locations between embankments and hardened structures such as floodwalls or closure structures. See EM 1110-2-2502 for guidance on tie-in transition areas.

9.3.10 Areas with long wind fetch lengths, the length of water which wind can blow without obstruction, may result in wind induced wave impacts. These impacts are typically of highest concern along wide rivers or river bypasses, lakes and for coastal levee systems.

9.4 Ways to Address Erosion PFMs.

9.4.1 Embankment geometry and materials play a critical role in the susceptibility of levees to erosion damage. The use of suitable soils to resist expected erosive forces is preferred for design. Availability of local borrow sources may limit the ability to use embankment materials that resist the expected velocity and shear stresses for a particular project. Using wider levee sections may be necessary in these circumstances that can allow for some erosion to occur but still allow for sufficient reliability that will not lead to breach. Any expected damage should be factored into the O&M requirements and long-term budget for the project. The preference would be to design armoring or vegetation to eliminate or reduce any expected damages when on-site embankment materials will not be able to resist the anticipated erosive forces.

9.4.2 Armoring of a levee is often required to minimize or eliminate erosion damage. A common and effective armoring method is placing different size riprap on levee slopes. There are other armoring techniques that designers may consider such as concrete slope paving, engineered revetment armoring, high performance turf reinforcement mats, etc. Figure 9-5 shows an example of a levee armored with riprap. Armoring may also be placed along the levee toe as a form of erosion protection, rock ballasted logs in the water in front of the levee and parallel to the levee toe is one option. Armoring may be required along the riverbank and bench to prevent erosion on the bank from encroaching on the levee foundation or embankment. The levee crest and/or landside of the levee may be armored to mitigate erosion due to wave action or overtopping flows. The presence of well-designed armoring can prevent or hinder the surface erosion significantly. Design guidelines for sizing and placing armoring (riprap protection) are available in EM 1110-2-1601. USACE Coastal and Hydraulics Laboratory Technical Report 10-7, "Flood-Side Wave Erosion of Earthen Levees: Present State of Knowledge and Assessment of Armoring Necessity" is a resource to review when considering armoring requirements to account for wave action.

Figure 9-5. Example of an Armored Levee.

9.4.3 Existing or planned vegetation must be considered when evaluating erosion failure modes and designing levees. Incorporation of trees or large shrubs within the levee access corridor as outlined in Chapter 1 must be evaluated during the design risk assessment (e.g. Phase 2). Including trees and large shrubs within the access corridor requires a design deviation that is supported by the risk assessment.

9.4.4 Other factors to consider include future meandering of the river channel and the potential to include structures beyond the levee to reduce likelihood of erosion impacting the levee foundation or embankment. It may be necessary to set the levee back from the river channel allowing additional batture between the channel and levee to account for future river meanders. Spur dikes can be design out from the riverbank to direct flow away from the levee foundation and/or embankment. A wave berm on the waterside of the levee may be constructed at the Stillwater level, the waves will break along the berm dissipating the wave energy before reaching the levee embankment.

CHAPTER 10

Levee Construction

10.1 Introduction. A properly constructed levee project is essential in achieving the requirements for both design and levee performance during and after construction. This chapter discusses requirements for the construction of new levees and modification of existing levees. These requirements should be considered during the design of the levee project and specified in the levee construction contract documents (e.g., construction drawings, specifications, and construction contract documents). These requirements discussed in this chapter include: the necessary preparation and treatment of the levee foundation for construction; construction of the levee embankment such as exploration trenches and fill selection, fill placement, and fill compaction; design considerations to improve levee embankment stability during construction; sequence and coordination of construction activities; construction quality control and quality assurance requirements for levee projects; and post construction documentation and risk assessments required for levee projects. Guidance for the plans and specifications for levee construction are further covered in ER 1110-2-1150.

10.2 Levee Foundation Preparation and Treatment.

10.2.1 General. Generally, levees are founded on soil foundations and the discussion on foundation preparation and treatment provided in this section applies to soil foundations. Minimum foundation preparation for levees consists of clearing and grubbing, and most levees will also require some degree of stripping. Clearing, grubbing, stripping, the disposal of resulting materials, and final preparation are discussed in the following paragraphs.

10.2.2 Clearing. Clearing consists of complete removal of all objectionable and/or obstructional matter above the ground surface. This includes all trees, fallen timber, brush, vegetation, loose stone, abandoned structures, fencing, and similar debris. The entire foundation area under the levee embankment, berms, and other levee project features should be cleared well ahead of the following construction operations.

10.2.3 Grubbing.

10.2.3.1 Grubbing consists of the removal, within the levee foundation area, of all stumps, roots, buried logs, pipes, foundation structures, old piling, old paving, drains, and other known objectionable matter. Roots or other intrusions over 1.5 inches in diameter within the levee foundation area should be removed to a depth of at least 3 feet below natural ground surface. Shallow tile drains sometimes found in agricultural areas should be removed from the levee foundation area. It is important that the contract documents identify any known objectionable materials that are not reasonably anticipated from the specified clearing operations. The contractor should anticipate removal of the root ball, large roots (greater than 1.5 inches in diameter), and the underground portion of stumps to a depth of 3 feet (or more depending on the tree type and size as necessary to remove large roots) where trees and stumps are visible above ground and removed as part of clearing operations. The decision to leave tree roots or stumps in place is generally discouraged. However, in some situations tree roots or stumps may be left in place after consideration of relative root ball or stump size, submergence and the rate of decay depending on wood species, and performance requirements for the levee.

10.2.3.2 Typically, the construction contractor is not required to investigate the entire site for buried objectionable materials that are not already identified or apparent; these features should be identified in the construction documents. Also, any buried debris associated with permitted or unpermitted landfill type deposits require special consideration in the contract documents and are not normally covered by a simple grubbing specification. The sides of all holes and depressions caused by grubbing operations should be flattened to a slope no steeper than 1 vertical to 1 horizontal (1V:1H) before backfilling. Backfill, consisting of material of similar nature to adjoining soils, should be placed in layers up to the existing foundation grade and compacted to a density at least equal to that of the adjoining undisturbed material. This will avoid "soft spots" under the levee and maintain the continuity of the natural blanket.

10.2.4 Stripping. After foundation clearing and grubbing operations are complete, stripping is commenced. The purpose of stripping is to remove low growing vegetation and organic topsoil. The depth of stripping is determined by local conditions and normally varies from 6 to 12 inches. Of this depth of stripping, 4 to 6 inches is usually adequate to remove the low lying vegetation and root systems. Additional stripping excavates and preserves the organic rich topsoil for future use. Stripping is required for the foundation of the levee embankment proper and may be needed under berms to avoid leaving a weak plane at the berm/foundation contact. All stripped material suitable for use as topsoil should be stockpiled for later use on the slopes of the embankment and berms. Unsuitable material must be disposed of by methods described in the next paragraph.

10.2.5 Disposal of Debris. Debris from clearing, grubbing, and stripping operations can be disposed of by burning in areas where this is permitted. The selected burn area should not be located within the footprint of any structural feature associated with the levee. After burning is complete, the ash should be buried or disposed of off-site according to all federal, state, or local regulations. When burning is prohibited by local regulations, debris needs to be disposed of in an environmentally approved manner. Disposal methods must be determined prior to contract bidding, and the contract specifications must clearly address disposal requirements.

10.2.6 Exploration Trench.

10.2.6.1 An exploration trench (often termed "inspection trench") should be excavated under all new levees unless special conditions as discussed later warrant its omission. The purpose of this trench is to expose or intercept any undesirable near-surface foundation features such as old drain tile, active or abandoned water or sewer lines or other utilities, animal burrows, buried logs, pockets of unsuitable material, or other debris. Inspection of the exploration trench allows the design engineers to assess the near-surface foundation conditions directly beneath the levee regarding impacts to the levee project requirements and to compare these findings to the results of the levee project's subsurface exploration. The trench should be located at or near the centerline of conventional fill levees or at or near the riverside toe of sand levees to connect with waterside impervious facings. See section 10.8 for information regarding the minimum documentation requirements for exploration trenches.

10.2.6.2 Dimensions and depth of the trench will vary with soil conditions and embankment configurations. Side slopes of the exploration trench should be stable and sufficient to allow personnel entry for inspection of conditions. Backfill should be placed only after a careful inspection of the excavated trench to ensure that seepage channels or undesirable material are not present; if they are, they should be dug out with a base of sufficient width to allow backfill compaction with regular compaction equipment. To backfill narrower trenches properly, special compaction procedures and/or equipment will be required. Trenches should have a minimum depth of 6 feet except when levee embankment height will be less than 6 feet, in which case the minimum depth should be equal to the embankment height or 3 feet, whichever is greater. Generally, it is preferred that the exploration trench is deeper than the typical depth of utility installation in the project area. If local practice dictates that utilities are typically installed deeper than 6 feet, the exploration trench should be dug to a depth 1 foot deeper than the typical utility installation depth. The bottom width of the trench should be wide enough to accommodate compaction equipment (tractor with towed roller or a self-propelled roller), which is generally a minimum of 8 to 12 feet. Depending on local conditions, exploration trenches may be eliminated if a seepage barrier or cutoff wall is constructed through the levee and into the foundation to a minimum depth of 6 feet into the foundation or deeper depths. Exploration trenches may be eliminated for modification to existing levees when conditions within the existing levee have been evaluated and there is not an expectation of adverse foundation conditions within the existing fill.

10.2.6.3 Contract specifications should address requirements for advance notification before trenches are excavated, to assure adequate opportunity for inspection and documentation by qualified personnel (typically provided by the owner of the levee construction project). After the trenches have been inspected and prior to backfilling, the trench subgrade should be scarified, moisture conditioned, and compacted. The required minimum relative compaction of the subgrade is typically less than the required trench backfill due to compacting upon native subsoil conditions. Trenches should be backfilled with compacted fill consistent in quality with the material that will be used in the overlying embankment. This work should be performed in a timely manner to limit exposure to possible rain events that could cause ponding and saturation of the foundation materials. If adverse foundation conditions that may impact the performance of the levee project are observed in the exploration trench, these conditions should be immediately brought to the attention of the designers or project engineer of the levee construction project before backfilling the exploration trench. The designer or project engineer for the levee construction project should determine the necessary actions to remedy the adverse foundation condition such that the requirements of the levee project are met.

10.2.7 Dewatering. Dewatering levee foundations for the purpose of excavation and backfilling in the dry is expensive. However, in order to meet the requirements of the levee project, a dewatering system for a levee construction project may be unavoidable after considering other design features. The cost factor in requiring dewatering system versus other design features may be an overriding consideration in choosing seepage control measures other than a compacted cutoff trench, berms, or riverside blankets. Where a compacted cutoff trench involving excavation or other required excavations below the water table must be provided, a properly designed and operating dewatering system is essential. TM 5-818-5 provides general guidance in dewatering system design. A dewatering system is generally designed to lower the water table a minimum of 5 feet beneath the work surface or excavation to prevent heaving at the base of the excavation, unstable excavation slopes, and lateral or vertical seepage from entering the excavation. A water table depth less than 5 feet may be used in certain situations. If the design of the dewatering system is the responsibility of the levee construction contractor, it is recommended that the contract specifications require a dewatering submittal for review and acceptance by USACE which describes how the Contractor will achieve the contract requirements, including type of system and locations of components.

10.2.8 Final Foundation Preparation. Unsuitable materials (i.e., soft or organic spots) in the levee foundation at or near the foundation surface should be removed and replaced with suitable compacted material. Suitable compacted material should be defined by the project delivery team to meet the requirements of the levee project. Except in special cases where foundation surfaces are adversely affected by remolding (soft foundations for instance), the foundation surface upon or against which fill is to be placed should be thoroughly scarified to a depth of at least 6 inches prior to the placement of the first lift of fill. This helps to ensure good bond between the foundation and fill and to eliminate a plane of weakness at the interface. The first lift of compacted fill is usually specified to be placed with a thinner loose lift thickness so that the initial lift and the scarified subjacent layer receive adequate compaction. The foundation surface should be kept drained and not scarified until just prior to fill placement in order to avoid saturation from rainfall.

10.2.9 Foundation Surface Acceptance.

10.2.9.1 Approval of the final foundation surface (after preparation and treatment) prior to placement of fill or concrete should be required for levee projects and pertinent features. Approval should be conducted by trained and experienced personnel, which is typically the project or resident engineer for the levee construction project. The designers of the levee project should also be involved in the approval of the final foundation surface to ensure that levee project requirements are met. The personnel and process to perform the approval should be captured in the engineering considerations and instructions for field personnel report. It may be determined that field personnel (i.e., quality assurance construction personnel or resident engineer) have the necessary training and expertise for standard levee construction projects to perform the approval. However, for levee construction projects with complex foundation conditions or requiring unique or unusual foundation preparation and treatment methods, design engineers and geologists may be required to perform approvals in addition to field personnel.

10.2.9.2 Foundation surfaces requiring approval should include surfaces that are to be covered by fill and/or concrete necessary for construction of the levee or other pertinent features. The foundation surfaces include but may not be limited to exploration trenches, working surfaces, and associated cut-slopes for the levee.

10.2.9.3 The method to conduct the approval of the final foundation surface will vary and should be scalable to the complexity of foundation conditions and the risks associated with the levee project. Visual observations by field personnel through quality assurance activities may be sufficient for simple levee foundation conditions (i.e., alluvial soil foundations, little to no utilities, etc). For complex foundation conditions (i.e., karst foundations, rock foundations with potential for defects and faults, numerous utilities or other man made features, etc), approval of the final foundation surface may be performed via formal inspections.

10.2.9.4 Visual observations and formal inspections generally include having the foundation surface photographed, mapping the locations of geological features and utilities, and as-built geometry of the foundation surface surveyed. If formal inspections are required, the specifications should include: a specific period of contract time for geologic and utility mapping by the approval personnel; a specific amount of construction contractor's staff and/or equipment time to assist in the cleanup of the foundation to allow for the mapping and/or the foundation inspection; a specific notification period and specific period of contract time for foundation inspection by approval personnel; and safety requirements for personnel performing the inspection. It may be advisable to proof roll the subgrade with a heavy piece of construction equipment to help identify soft/unstable soil conditions.

10.2.9.5 Approval of the final foundation surface should be documented by a final foundation approval report signed by necessary approval personnel. The final foundation approval report may include a summary of geologic conditions (significant geological features and water inflow), utilities encountered (types, sizes, and actions take to remove), and final foundation preparation (description of removal of/modification of foundation or slope support materials, description of scaling, washing, and cleaning, description of dental treatment applied, and description of groundwater seepage mitigation and control of standing/flowing water). Documentation of the formal inspection and approval should be included in the levee project documentation (see Section 10.8).

10.3 Levee Embankment Construction.

10.3.1 Fill Material Selection.

10.3.1.1 The required properties of levee fill depends on the strength, hydraulic conductivity, consolidation, erosion, and other fill characteristics necessary to satisfy the levee performance requirements. Soil type, placement methods, and compaction requirements (i.e., compaction effort and moisture content) for levee fill should be specified to satisfy the levee fill properties. Suitable fill material for levees can range from clays, silts, sands, or a combination thereof. Table 10-1 provides a list of suitable fill materials properties for typical levee construction. The selection of borrow areas for levee fill is discussed in Chapter 4.

10.3.1.2 Generally, levee embankments are constructed as a homogeneous levee cross-section. Zoning is an alternative in which backfill materials with different properties are placed in different zones such that the levee section is not homogeneous throughout the section. However, zoning is usually neither necessary nor practicable due to the size and length of the levee. However, in some circumstances, zoning of the levee embankment may be necessary due to limitations of available fill material or when modifying an existing levee that consists of fill more pervious or less pervious than the new levee fill. In this case, more impervious material should be placed toward the waterside of the embankment and the more pervious material toward the landside slope. This will help prevent buildup of seepage pressures in the levee embankment that could otherwise cause instability of the landside levee slope. If zoning is contemplated, material requirements (i.e., material properties, compaction requirements, etc.) for each zone and zone dimensions must be identified in advance and clearly defined in the contract documents. Additional controls and testing may be necessary to assure desired intent is met. Constructability and cost impacts of zoned levee embankments must be considered before implementation.

10.3.1.3 Filters in the levee embankment may be required due to concerns with internal erosion due to seepage through the levee embankment. In addition, drains may be required to intercept foundation and embankment seepage. Filters and drains should be designed and constructed to meet current standards to prevent internal erosion and control drainage. Filters and drains should be designed and constructed such that proper drainage (such as a seepage collection system) is provided to prevent buildup of excess seepage pressures. Appropriate soil cover over filters and drains should be provided such that heave conditions do not occur if the filter or drain becomes clogged. If perforated pipes, well screens, or similar items are utilized in filters or drains to collect and discharge water, maintenance access locations (such as a cleanout) should be provided so that these items can be maintained over the life of the levee project.

Table 10-1. Typical Suitable Fill Material for Levee Construction.

Suitable Fill Material Properties	Reason
Plasticity index of fine-grained fill should be less than 40 but greater than 8 using the ASTM method.	High plasticity clays within the levee slopes are likely to lead to slough slides. If high plasticity fine-grained fill is used, refer to Chapter 7 for proper slope design.
Organic content should be less than 9%.	High organic content may result in low fill density, low shear strength, and increased fill settlement
Unsuitable material such as organic material (roots, grass, weeds, and sticks), frozen material, large rocks (greater than 3 inches in diameter), trash, and other debris should be avoided.	Unsuitable material may lead to potential seepage paths through the levee and poor compaction of the levee fill.
Dispersive clays should be avoided.	Dispersive clays are highly erodible and very susceptible to internal erosion.
Clayey sands intended for impervious fill should have greater than 30% fines by weight passing the No. 200 sieve and a minimum plasticity index of 8.	Ensure low hydraulic conductivity.
Pervious fill for sand-fill levees or pervious zones (i.e., drains) should contain less than 5% fines by weight passing the No. 200 sieve after placement and compaction.	Ensure material is free draining and does not sustain a crack.
Pervious zones (i.e., drains) should not contain calcareous sands and/or gravels.	Degradation of these materials can lead to cementation and clogging, and overall reduction of filter/drain capacity over time.

10.3.2 Compaction Requirements.

10.3.2.1 Fill strength, hydraulic conductivity, consolidation, erosion and other soil behavior characteristics are directly related to the fill compaction effort (e.g., lift thickness and compactive energy level) and fill moisture content during compaction (i.e., in relation to the optimum moisture content). Thus, the selection of compaction requirements should satisfy the suite of required levee fill properties. The specified compaction requirements for levee fill should include maximum loose lift thickness, minimum relative compaction/relative density (specified in relation to standard compaction tests), and range of allowable moisture content.

10.3.2.2 Fill compaction requirements of impervious and semipervious materials are divided into three categories as shown in Table 10-2: fully compacted, semi-compacted, and minimally compacted. Levee embankments composed of impervious and semipervious materials (i.e., materials with compaction characteristics that produce a well-defined maximum density at a specific optimum water content) typically require fully compacted or semi-compacted compaction requirements. Waterside and landside berms (for seepage or stability purposes) may be constructed using semi-compacted compaction requirements. The compaction requirements shown for each category are the minimum requirements and should be adjusted as necessary to achieve the required levee fill properties. For example, if ductility in the levee embankment is important to limit the effects of differential settlement or lateral movement, then semi-compacted compaction requirements may be preferred over fully compacted.

10.3.2.3 Moisture content of the levee fill during compaction has a direct impact on the fill behavior characteristics. Generally, moisture content is specified as a range in relation to the optimum moisture content based on standard compaction tests (e.g., ASTM D698). For levee construction, it is preferred to compact levee earth fill wet of the optimum moisture content as this results in lower permeabilities, higher erosion resistance, and more ductility to prevent cracking during settlement. As discussed in Chapter 4 (Borrow Areas), moisture conditioning of the fill material may be required to achieve the desired moisture content during compaction.

10.3.2.4 Pervious Levee Fill.

10.3.2.4.1 Pervious levee fill consisting of sands or sands and gravels are sometimes used for levee construction that are placed with a combination of normal earth moving equipment and hydraulic fill methods. Compaction of pervious levee fill can be controlled by either a relative density or a relative compaction specification. Chapter 3 outlines the pros and cons of the use of relative density or relative compaction for control of compaction of cohesionless soils. For a relative density specification, it is recommended that sand embankment materials be compacted to a minimum of 70 percent relative density. The required relative density should be increased if necessary for the design (e.g., if a higher unit weight or strength is required, or the levee is in a seismically active zone) up to a maximum of 80 percent. For a relative compaction specification, it is recommended that sand fill materials be compacted to a minimum density of 95 percent of the maximum dry density determined from ASTM D1557, ASTM D4253 or ASTM D7382.

10.3.2.4.2 A more complete discussion of the design considerations for hydraulic fill levees is included in Section 7.3.2. Hydraulic fill levees should be constructed with sands having less than 5 percent fines. Compaction control for hydraulic fill levees has historically been constructed with use of relative density, although relative compaction could also be specified. Hydraulic fill levees are typically constructed with very flat slopes (1V:4H waterside and 1V:5H landside) and are not used in seismically active areas.

10.3.2.4.3 Where underwater placement is required, it can best be accomplished with pervious fill using end-dumping, dragline, or hydraulic means, although fine-grained fill can be so placed if due consideration is given to the low density and strength obtained using such materials. Details of placement by end-dumping underwater are included in Section10.4.3.

Table 10-2
General Compaction Categories for Levee Construction[1].

Category	Density Requirement	Lift Thickness	Moisture Control	Compaction Equipment	Recommended Testing Frequency[4]	Use
Fully compacted	≥ 95% Maximum Dry Density[2]	6 to 9 inch maximum loose thickness	Depends on soil type and the required fill properties. Generally requires a more restrictive moisture control with respect to standard effort optimum water content[2].	Tamping rollers (including self-propelled), rubber tired rollers, crawler-type tractors, or sheep foot rollers meeting required equipment specification.	Maximum dry density[2] and optimum moisture content[2]: Minimum one test for every 20,000 to 25,000 cubic yards of fill placed Field density[3] and field moisture[3] of the compacted fill: Minimum one test for every 1,000 cubic yards of fill placed.	Levee embankment requiring high shear strengths and low compressibility.
Semi-compacted	≥ 90% Maximum Dry Density[2]	9 to 12 inch maximum loose thickness	Depends on soil type and the required fill properties. Moisture content range is wider than fully compacted category and may be closer to the in-situ moisture content of the fill material.	Tamping rollers (including self-propelled), rubber tired rollers, crawler-type tractors, or sheep foot rollers meeting required equipment specification.	Maximum dry density[2] and optimum moisture content[2]: Minimum one test for every 20,000 to 25,000 cubic yards of fill placed Field density[3] and field moisture[3] of the compacted fill: minimum one test for every 1,000 cubic yards of fill placed.	Levee embankment, berms, and levee ramps not requiring high shear strengths or founded on soft soil. This is to reduce the weight of the foundation to limit settlement.
Minimally compacted	≤90% Maximum Dry Density[2]	Fill cast or dumped in place in thick layers	Some moisture content control but typically only needed to improve workability of the fill material.	Hauling and spreading equipment	Little to no testing requirements	Not for use in levee embankments. Mainly used for filling of pits and temporary emergency fill.

[1] Information provided in this table is from typical specifications. For each levee project, construction requirements should be clearly defined in the contract documents based on project-specific considerations.
[2] Maximum dry density and optimum water content for most levee projects are based on Standard Proctor Tests (ASTM D 698).
[3] Field Density Tests per ASTM D6938 (Nuclear Method) or ASTM D1556 (Sand Cone Method). Field Moisture Tests per ASTM D2216 (Oven Dry Method), ASTM D4643 (Microwave Method), or ASTM D6938 (Nuclear Method). Nuclear and microwave methods should be verified with other methods per Section 10.6.5.
[4] Testing requirements shown assume that the fill material type is consistent. If the fill material type varies, maximum density and optimum moisture content tests should be conducted regardless of volume of fill placed. Field density and field moisture tests on compacted fill should be conducted on each lift regardless of volume of fill placed.

10.3.3 Soil Stabilization. Soil stabilization of the levee embankment fill may be required to meet the levee project requirements and borrow source limitations. Commonly used soil stabilization for levee embankments are cement stabilization and lime stabilization. Stabilization with industrial by-products is generally not recommended due to poor or variable physical and chemical properties of the by-products and potential for public health and environmental hazards. A brief discussion on cement and lime stabilization is provided below.

10.3.3.1 Cement Stabilization. Cement stabilization may be used for slope protection of levee embankments. Cement stabilization (or sometimes referred to as soil cement) is produced by blending, compacting, and curing a mixture of soil/aggregate, portland cement, possibly admixtures including pozzolans, and water to form a hardened material with specific engineering properties. The decision to use cement stabilization for slope protection versus other conventional slope protection methods depends on cost and required performance. In general, cement stabilization can be used on most soils of medium to low plasticity. However for levee embankment protection, better quality granular materials are recommended since the soil cement may be subjected to repeated cycles of wetting-drying, freezing-thawing and wave action. Refer to Appendix I for more discussion on the use and design of soil cement for levee slope protection.

10.3.3.2 Lime Stabilization. Lime stabilization may be used to stabilize slopes of levee embankments composed of high plasticity clays (CH) and is also sometimes used to mitigate dispersive clay behavior. Depending on the application and project-specific requirements, lime stabilization may only be required to the depth of expected soil moisture changes due to seasonal wetting and drying cycles. Generally, hydrated lime conforming to ASTM C977, type N (normal finishing hydrated lime commercial grade) should be used in lime stabilization applications. Chimney flue by-products are not recommended for lime stabilization due to their variability in effectiveness and quality. Percentage of lime (by dry unit weight) required for stabilization is usually selected based on laboratory testing on soil samples mixed with varying percentages of lime. As the plasticity of the soil increases, the percentage of lime required for stabilization generally increases. If the lime percentage required for stabilization is 6 percent or more by dry weight, a double application of lime may be required. The first application of lime consists of applying half of the required lime percentage, mixing the soil and lime with a rotary pulverizer, and sealing the surface with a flat steel roller to prevent drying out of the soil-lime mixture. The second application of lime is typically applied after 24 to 48 hours of curing is allowed since the first application. The second application applies the remainder of the required lime percentage. Additional moisture may be needed during the second application to ensure enough moisture is available to support the hydration process.

10.3.4 Special Considerations for Existing Levee Embankments.

10.3.4.1 Temporary Degrading. Special construction considerations should be given to rehabilitation or modifications of existing levee embankments. Temporary degrading (if required) of an existing levee embankment for the construction of the levee rehabilitation or modification poses an increased likelihood of the levee to overtopping and inundation of the leveed area. For leveed areas with high consequences, a risk assessment may be required on the proposed levee degrading. Consideration should be given to the use of cofferdams or other temporary flood mitigation measures if the risk assessment shows that additional height (above the degraded levee) is required. If a cofferdam or other temporary flood mitigation measure is not required, the length of levee degraded at one time should be limited and should be based on the accuracy (i.e., timeliness and reliability) of flood forecasts and the contractor's ability to replace the degraded levee portion prior to flood inundation.

10.3.4.2 Fill Placement. Placing fill on an existing levee embankment generally requires benching into the slope of the existing levee (Figure 10-1) in order that the new levee embankment fill is placed and compacted in horizontal layers. Benching into the existing levee slope also prevents the development of a slide surface between the new fill and existing fill. The vertical face of the excavated bench in the existing levee should generally be a minimum of 1 foot in height but should generally not exceed 2 feet in height. The average slope of the benched excavation should be no steeper than 1V:2H. It is a good practice to include a key at the toe of the slope typically 10 feet in width and 2 feet in depth.

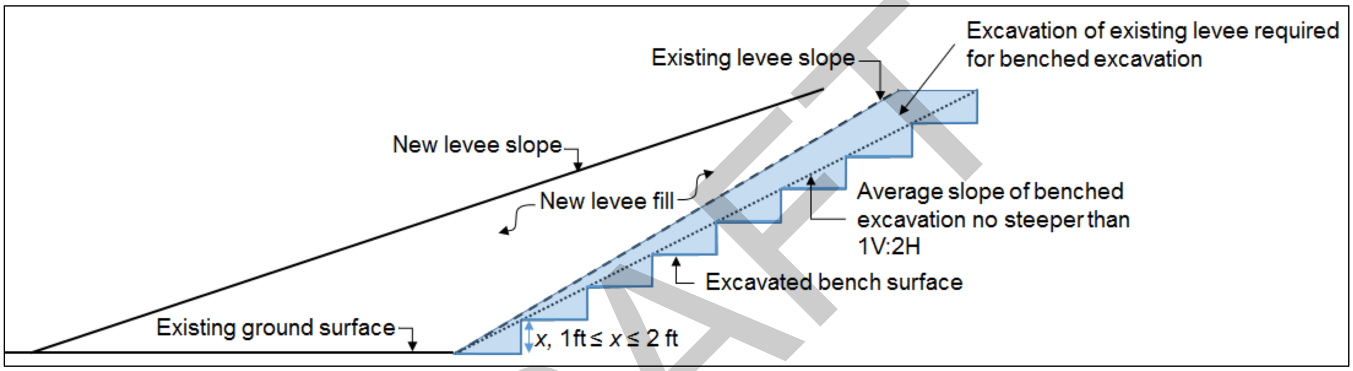

Figure 10-1. Sketch of a Benched Excavation.

10.3.4.3 Seepage Barriers through the Levee Embankment. Construction of seepage cutoff walls through a levee embankment requires a construction platform (may be up to 30 to 40 feet wide to accommodate a hydraulic excavator or mix-in-place equipment or a minimum of 45 feet wide to accommodate panel construction equipment) achieved by either partial degrading of the levee or placement of new fill to construct the platform. When a cutoff wall is used that has potential long term settlement characteristics (such as a soil-bentonite cutoff wall) the top of cutoff wall should be encapsulated within impervious materials. If the embankment is considered impermeable, no additional encapsulation is required. However, if the embankment is considered permeable, the top 2 to 3 feet of the degraded surface (minimum 8 foot width) should be excavated and replaced with impervious fill. The reconstructed levee embankment above the cutoff wall should consist of an impervious layer or impervious cap. Design and construction requirements for seepage barriers and cutoff walls are provided in the forthcoming USACE manual on cutoff walls.

10.3.5 Use of Geosynthetics in Levee Construction.

10.3.5.1 The Unified Facilities Criteria (UFC) 3-220-08FA, Engineering Use of Geotextiles, provides guidance on acceptable use of geotextile in water retaining structures such as levees or other similar flood risk management structures. Geotextile applications and functions include separation, drainage, filtration, reinforcement, containment, and protection.

10.3.5.2 The use of geotextiles for drainage and filtration applications in levees that pose a significant threat to life safety or potential threat for high economic or environmental consequences should be avoided if possible or used with caution including a full understanding of the risks associated with performance of geotextiles. Ancillary features associated with levees (such as pavement and landscaping applications) generally do not significantly affect the levee performance and for these features, geotextiles are generally acceptable for use. For drainage and filtration applications, the concern associated with the performance of geotextiles and geosynthetics is clogging. During a workshop in October 2000, the National Dam Safety Review Board concluded that geotextiles and geosynthetics should not be used within a dam where they are both critical for dam safety and inaccessible for repair or replacement. Furthermore, the National Dam Safety Review Board also concluded that geotextiles and geosynthetics can potentially be used in locations that are critical for dam safety but accessible for replacement.

10.3.5.3 The U.S. Army Corps of Engineers, New Orleans District (MVN) has successfully used geotextiles as reinforcement in the construction of levees founded on soft soils in the Hurricane and Storm Damage Risk Reduction System (HSDRRS). Based on research and experience gained over years of use, MVN has adopted a design procedure for geotextile reinforced levees. This design procedure evaluates the global stability of the reinforced levee embankment, bearing capacity of the foundation beneath the levee, the required embedment/anchorage length, lateral embankment sliding/spreading along the top of the geotextile, creep and long term embankment stability, and consolidation and settlement. Further discussion of geotextile reinforcement is discussed in Section 10.4.6.

10.4 Methods of Improving Levee Stability During Construction.

10.4.1 General. Levees located on foundation soils that cannot support the levee embankment because of inadequate shear strength require some type of foundation treatment if the levee is to be built. Foundation deposits that are prone to cause problems are broadly classified as follows: (1) very soft clays, (2) sensitive clays, (3) loose sands, (4) natural organic deposits, and (5) manmade debris (for example, trash site). Very soft clays are susceptible to shear failure, failure by spreading, and excessive settlement. Sometimes soft clay deposits have a zone of stronger clay at the surface caused by desiccation. Even if stronger clays at the surface provide adequate shears strength, expensive foundation treatment may be necessary to mitigate for long term settlement. Sensitive clays are brittle and even though they possess considerable strength in the undisturbed state, they are subject to partial or complete loss of strength upon disturbance. Fortunately, extremely sensitive clays are rare. Loose sands are also sensitive to disturbance and can liquefy and flow when subjected to shock or even shear strains caused by erosion at the toe of a slope. Most organic soils are very compressible and exhibit low shear strength. The behavior of debris deposited by humans, such as industrial and urban refuse, is so varied in character that its physical behavior is difficult, if not impossible, to predict. The following paragraphs discuss methods of dealing with foundations that are inadequate for construction of proposed levees.

10.4.2 Excavation and Replacement. The most direct method of dealing with excessively compressible and/or weak foundation soils is to remove them and backfill the excavation with suitable compacted material. This procedure is feasible only where deposits of unsuitable material are not excessively deep. Excavation and replacement should be used wherever economically feasible.

10.4.3 Displacement by End Dumping.

10.4.3.1 Displacement by the end dumping method for levee construction is generally discouraged due to the concerns on whether a reliable levee can be constructed using this method. End dumping is completed by dumping new fill material at the continuously progressing end of a new embankment. Displacement by end dumping can result in soft soils along levee/foundation contact leading to levee instability and excessive differential settlement, or can result in a high permeability zone along levee/foundation contact that could result in internal erosion failure modes.

10.4.3.2 There are unique situations where local experience has demonstrated displacement by end dumping can provide an effective method to construct a reliable levee. These situations occur for low levees that cross a slough or stream channel that have very soft fine-grained soils (often having high organic content) along the bottom or where a working platform is needed to construct a low levee on soft soils. The depths of these very soft soil deposits may not be large and a levee of adequate stability can be obtained by end-dumping fill from one side of the slough or channel, pushing the fill over onto the soft materials, and continually building up the fill until its weight displaces the foundation soils to the sides and front.

10.4.3.3 Before using displacement by end dumping for the levee project, considerations should be given during design to whether short and long-term material properties (e.g., shear strength, unit weight, compressibility, and hydraulic conductivity) along the levee/foundation contact will result in the required levee reliability. Internal erosion modes along interfaces between end-dumped materials, compacted materials and foundation materials should be considered and mitigated as necessary. In addition, displacement by end dumping often results in challenges in construction contracts as estimating, bidding, measurement, and payment for this type of construction can be very hard to quantify.

10.4.3.4 The placement of fill using the end-dumping method should be advanced with a V-shaped leading edge so that the center of the fill is most advanced, thereby displacing the soft material to both sides. A wave of displaced foundation material will develop (usually visible) along the sides of the fill and should be shaped to drain by grading but not removed. Where the end-dumping method is used to provide a working platform on soft foundation soils, only enough fill material should be hauled in and dozed onto the foundation to build a working platform or pad upon which the levee embankment can be built by conventional equipment and methods. Material forming the working platform should not be stockpiled on the platform since this may result in a shear failure through the platform and underlying foundation. Only small bulldozers should be used to spread and work the material. Where the foundation is extremely weak, it may be necessary to use a small clamshell to spread the material by casting it over the area.

10.4.4 Staged Construction.

10.4.4.1 Staged construction refers to the building of an embankment in successive physical stages or intervals of time. This method is used where the strength of the foundation material is inadequate to support the entire weight of the embankment, if built continuously at a pace faster than the foundation material can consolidate and increase in shear strength. Using this method, the embankment is built to intermediate grades and allowed to rest for a time before placing more fill. Such rest periods permit dissipation of pore water pressures which results in a gain in strength so that higher embankment loadings may be supported. This approach is advantageous when pore water pressure dissipation is reasonably rapid because of foundation stratification with short drainage paths. This procedure works well for clay deposits interspersed with highly pervious silt or sand seams. However, such seams must have exits for the escaping water otherwise they themselves will become seats of high pore water pressure and low strengths (relief well points can be installed on the landside to increase the efficiency of pervious layers in foundation clays). Initial estimates of the time required for the needed strength gain can be made from results of consolidation tests and study of boring data. Piezometers should be installed during construction to monitor the rate of pore water dissipation, and the resumption and rate of fill placement should be based on these observations, together with direct observations of fill and foundation behavior. Disadvantages of this method are the delays in construction operation, and uncertainty as to its scheduling and efficiency.

10.4.4.2 If the expected rate of consolidation under staged construction is unacceptably slow, it may be increased by the use of prefabricated vertical (wick) drains. Such drains are geotextile wrapped plastic cores that provide open flowage areas in the compressible stratum. Their purpose is to reduce the length of drainage paths, thus speeding up consolidation. They can be pushed into place through soft soils over 100 feet deep. Before the drains are installed, a sand drainage blanket should be placed on the foundation which serves not only to tie the drains together and provide an exit for escaping pore water, but as a working platform as well. This drainage blanket should not continue across the entire base width of the embankment to prevent a continuous seepage path at the base of the levee. Special embankment zoning or seepage cutoffs may be required through a wick drainage blanket to minimize the potential for a continuous seepage path. Potential levee underseepage impacts due to the use and locations of wick drains should be considered and appropriately addressed in the design. See Chapters 7 and 8 for additional information on staged loading, wick drains, and prefabricated vertical drains.

10.4.5 Densification of Loose Sands. The possibility of liquefaction of loose sand deposits in levee foundations may have to be considered for critical components in levee projects where seismic hazards are likely to impact the levee reliability. Risk-informed decision making (Chapter 1) should be utilized when considering seismic hazards and seismically initiated failure modes. Methods for densifying sands, such as vibroflotation, are generally very costly. Therefore, design features to improve levee reliability should consider increases in the levee section width, such as wider levee crest, berms, and flatter slopes.

10.4.6 Geosynthetic Reinforcement. Geotextile and geogrid reinforcement has been successfully used to improve the stability of levees founded on soft and highly compressible soils. The design of geosynthetic reinforcement considers numerous factors such as global stability of the reinforced levee embankment, bearing capacity of the foundation beneath the levee, the required embedment/anchorage length of the geosynthetic, lateral embankment sliding/spreading along the top of the geotextile, creep and long term embankment stability, and consolidation and settlement. The geosynthetic reinforcement are generally located near the base of the levee and are often more efficient when using a single layer of reinforcement. The strength of the geosynthetic reinforcement is generally selected by the tensile strength at a strain level that will minimize creep (typically at 5% axial strain) and at a strain level compatible with the strain at peak shear strength of the soil. Refer to Appendix H for design methodology and additional construction considerations for geosynthetic reinforced embankments on soft foundations.

10.4.7 Stability Berms. Stability berms provide resistance against levee slope stability related failures by providing weight (main benefit) and shear resistance. Stability berms are generally constructed of earthen material placed against the levee embankment slope and extend beyond the levee toe. A drainage layer at the levee and berm interface may be specified to safely filer and dissipate seepage from the levee embankment. It is generally preferred that stability berms be more pervious than the levee and underlying natural material to prevent excess build of pore pressures during flood events or drawdown situations. In some situations, a drainage layer at the levee and berm interface may be specified to safely filter and dissipate seepage from the levee embankment. However, the stability berm should be constructed of materials that provide the required strength and hydraulic conductivity necessary to meet the requirements of the levee performance. Surficial erosion of the berms will impact the berm's function and should be incorporated into the berm design and selection of berm material.

10.5 Engineering Considerations During Construction.

10.5.1 Engineering During Construction. Levee construction activities vary based on project-specific requirements. Thus, levee project requirements should be communicated clearly and comprehensively in the design/bid package to the contractor and routinely reviewed during the levee construction activity. Successful levee construction requires a coordinated team effort including the project delivery team (PDT), construction quality inspectors, local sponsors, and the contractor. The PDT consists of engineers, environmental specialists, and project managers who developed the levee design/bid package. As specified in ER 1110-1-12 (Quality Management), key members of the PDT should be involved during the levee construction activity. The involvement of the PDT members is scalable to the complexity of the construction activity and risks associated with the levee project. Generally, for levee projects, the lead engineer responsible for the design of all geotechnical related aspects of the levee project should become an indispensable member of the construction team and should attend all pre-construction meetings; observe construction processes; assist in developing contract modification requests; and evaluate transmittals, Quality Control (QC) reports, Quality Assurance (QA) reports, and contractor requests for information (RFIs). In addition, other PDT members should be involved who represent engineering disciplines associated with the construction activity. Construction site visits by pertinent PDT members (i.e., engineers) are recommended and may be necessary on complex levee projects at critical points of construction. The engineering PDT members should be informed and have reviewed all construction changes to ensure the changes will not impair the quality and functionality of the design, increase risks to the public and environment, cause a safety or environmental hazard, or create an unsatisfactory condition. Submittals and shop drawings should also be reviewed by the engineering PDT members.

10.5.2 Instructions to Field Personnel.

10.5.2.1 Prior to administering a levee construction contract, the design PDT should prepare a brief report on the engineering considerations and instructions for field personnel to aid them in the supervision and inspection of the construction contract. The report should summarize pertinent design information and include informal discussions on why specific designs, material sources, construction plant locations, etc. were selected. This information will assist field personnel by providing the insight and background needed to review construction contractor proposals and resolve construction problems without compromising the design intent. The considerations and instructions must not conflict with the contract drawings and specifications. The report shall be reasonably short and organized for quick reference in field situations. Engineering Regulation ER 1110-2-1150 (Engineering and Design for Civil Works Projects) provides an outline to aid in preparing the engineering considerations and instructions for field personnel. In addition, the report should specify construction milestones and features that require site inspections by the PDT. The number of required site inspections will vary based on the complexity of the levee project but typically include foundation inspections (including exploration trench inspections) and initial fill placement. The report should also include a discussion on the submittal register, PDT members who will review each submittal (if necessary), and why review of the submittal is pertinent to the levee project. Site visits with both the design PDT and construction quality inspectors is typically required to clarify any issues affecting the construction, including aesthetic considerations, which cannot be conveyed via the report on engineering considerations and instructions for field personnel.

10.5.2.2 The observational method described by Ralph Peck (Terzaghi et al. 1996) and discussed in Section 11.15.1 should be implemented during construction of the levee project. Implementation of the observational method should be scaled to the complexity of the levee construction project and potential flood risks associated with the levee construction project. For pertinent levee project features, this method includes assessment of the most unfavorable conceivable conditions in addition to the expected conditions or design assumptions. A monitoring plan should be developed by the design PDT regarding specific measurements to be taken as construction proceeds. Estimates should be performed on measurements for the expected conditions and the most unfavorable conditions. Parameters measured include but are not limited to detailed strength, consolidation, and hydraulic conductivity as well as detailed survey information regarding movements or settlements to verify design assumptions or expected values. These measures are generally not included in quality control (construction contractor) or quality assurance (levee contract owner) responsibilities. Thus, implementation of the observational method will require additional engineering resources. Courses of action should be prepared to offset the construction deficiencies when actual measurements deviate from the expected measurements. Information obtained through the observational method should be documented and included in the design documentation report (See Section 10.8.1) for the levee.

10.6 Construction Quality Control and Quality Assurance.

10.6.1 Introduction. Construction operations may be carried on concurrently along many miles of levee. This means that more time and manpower are needed to cover the operations on many levee jobs. Levee construction is impacted by high river stages during construction. In addition to potential for flood impacts to partially completed levees, borrow areas are often inundated during higher river stages, even before reaching flood levels. This can result in extended periods where construction is not feasible, resulting in longer durations to complete construction.

10.6.2 Quality Control.

10.6.2.1 Per ER 1180-1-6, quality control (QC) is generally the responsibility of the construction contractor and is the process by which the contractor ensures and reports that the requirements of the contract are achieved. The owner of the levee construction project (either USACE or the local sponsor) should establish the requirements for the contractor QC. It is critical that the contract contain specific requirements for types and frequency of control testing and reporting, as well as qualifications of those performing the tests. It is often appropriate to require control testing be performed by an independent lab, whose qualifications to perform the specified tests have been validated in accordance with ER 1110-1-8100. In some cases, it may be considered acceptable for the contractor to self-perform some control tests, but qualifications and validation requirements must still be specified in the contract documents.

10.6.2.2 A QC plan should be prepared by the construction contractor prior to commencement of construction activities and should consist of plans, procedures, and contractor quality control organizational governance necessary to produce an end product which complies with the contract requirements. The plan should cover all construction operations, both on-site and off-site, and should be integrated with the proposed construction sequence. The owner of the levee construction project will review and approve the contractor QC plan. QC testing reports are generated as work progresses and compiled at the completion of the project. The reports should include all compaction density tests and field testing control data, comparison testing, calibrations, and individual field test reports with spatial coordinates and elevations of tests. All tests results should be kept in a database (such as a spreadsheet) such that any failed test can be easily cross referenced to passing retests.

10.6.3 For levee projects that pose significant threat to life safety, it is generally preferred that contractor QC inspections and test results are certified by a licensed professional engineer or professional geologist registered in the state who has experience and knowledge related to the levee construction and testing activity. Certifications by the engineer typically require that the tests and observations be performed by or under the direct supervision of the engineer and test results are representative of the materials or conditions being certified. These certifications should be documented in a report signed by the professional engineer. The certification by a registered professional engineer in the practice of professional engineering constitutes an expression of professional opinion regarding those facts or findings which are the subject of the certification.

10.6.4 Quality Assurance.

10.6.4.1 Per ER 1180-1-6, Quality Assurance (QA) is required on levee construction projects and is generally the responsibility of the owner of the levee construction project (either USACE or the local sponsor). QA evaluates whether the construction contractor is performing the duties of Contractor Quality Control as defined in the contract and whether the levee construction project requirements are met. Prior to construction, a QA plan should be developed by the owner of the levee construction project and should be updated throughout the levee construction project as necessary. The QA plan should contain requirements for types and frequency of QA testing to assure reliability and acceptable accuracy of the contractor's testing program. QA testing should be also performed by a USACE-validated lab (see ER 11110-1-8100) that is independent of the construction contractor.

10.6.4.2 Quality assurance tests should be performed by the contract management organization (USACE or the local sponsor) to assure acceptability of the completed levee work. The amount of quality assurance engagement and testing should be scalable to the scope and potential risk posed by the levee project. Generally, QA tests should be performed at a frequency of 10 to 25 percent of the contractor QC testing in order to verify the QC test procedures and results. Lower frequency of quality assurance testing represents demonstrated confidence in the QC procedures. Thus, for budgeting purposes, the contract management organization may assume 25 percent and then reduce in practice as the QC procedures demonstrate technical competence and results.

10.6.5 Compaction Testing.

10.6.5.1 Typical frequency of field density and field moisture content tests are provided in Table 10-2. Selection of the appropriate compaction control curve should be based on the borrow source location, soil classification, and Atterberg limit tests. Atterberg limit tests may be performed on every tenth moisture test (more or less depending on the project and varying conditions). Atterberg limit tests may also be performed for each density field test to ensure that the appropriate soil compaction test is compared to the density field test. Field in-place density (e.g., dry density) may be evaluated in accordance with ASTM D6938 (nuclear method) or ASTM D1556 (sand cone method). If the nuclear method is used for field density testing, the sand cone method should be used to verify the accuracy of the nuclear method.

10.6.5.2 Where density is critical to levee performance, it is recommended that a correlation be established early in the project and that at least one adjacent sand cone test be performed adjacent to every fifth nuclear density test. If field density (dry density) determined by the nuclear method varies by more than 2 pounds per cubic foot in comparison to density determined by sand cone tests, and are consistently high or low, adjustment of the calibration curve should be required. The accuracy and recalibration (if necessary) of the nuclear method should be verified throughout the construction of the levee project.

10.6.5.3 Moisture content tests are generally performed in accordance with ASTM D2216 (oven dry), ASTM D4643 (microwave method), or ASTM D6938 (nuclear method). The accuracy of the nuclear method and microwave method are affected by certain minerals and soil types. If the nuclear or microwave method is used for moisture determination, the oven dry method should be used to verify the accuracy of these methods. A minimum of five comparison tests may be performed at the start of construction and corrections of these methods may be required. If nuclear or microwave method results are within 3 percent of results from the oven dry method, no correction of the moisture is generally required. The nuclear and microwave moisture methods should be verified throughout the project with rates of around one oven moisture for every 10 nuclear and/or microwave tests. Note that for nuclear methods, calibration checks of both the density and moisture gages should be made on each different type of material encountered.

10.6.5.4 A compaction report should be prepared for construction of levee embankments or other earth fill that constitutes the levee alignment by the levee construction contractor and signed by the professional engineer (generally preferred). The compaction report should contain the test results, test locations, limits of certified fill, and a statement that certifies that all fills were constructed in accordance with the plans and specifications. The levee construction contractor should establish and document spatial coordinates and elevations of locations for all compaction test (e.g., density tests, moisture tests). The contractor should prepare drawings showing locations of the quality control density tests and dimensions of compacted fill. The report is generally submitted on the workday following the test.

10.6.6 Potential Construction Deficiencies. Quality control and quality assurance is necessary to prevent construction related deficiencies that will impact the levee performance during and after construction. A list of possible construction deficiencies and related consequences to the levee performance are provided in Table 10-3.

Table 10-3
Possible Construction Deficiencies.

Deficiency	Consequence
Organic material not stripped from foundation	Differential settlements; shear failure; internal erosion caused by through-seepage or under-seepage
Highly organic or excessively wet or dry fill	Excessive settlements; inadequate strength; high permeability zones; stability issues (i.e., slides)
Placement of pervious layers extending completely through the embankment	Allows unimpeded through-seepage which may lead to internal erosion and failure
Inadequate compaction of embankment (lifts too thick, haphazard coverage by compacting equipment, incorrect moisture content, etc.)	Excessive settlements; inadequate strength; through-seepage due to stratification of the embankment
Inadequate compaction of backfill around structures in embankment	Excessive settlements; inadequate strength; provides seepage path between structure and material which may lead to internal erosion and failure by piping
Inadequate processing of lifts prior to compaction and/or improper scarification between lifts	Stratification; inconsistent density or voids; slope stability issues; internal erosion caused by through-seepage
Seasonal shutdown layers not properly treated or placement in freezing weather	Pervious layer through the levee embankment; differential settlement of the overlying embankment leads to transverse cracking in the levee embankment

10.7 Sequence and Coordination of Construction Activities.

10.7.1 The sequence and coordination of construction activities are typically specified in the special clauses section of the construction contract documents. The special clauses section should be used to the full extent to control the sequence of contractor's operations and to include one-of-a-kind provisions necessary to meet all USACE, sponsor, and stakeholder requirements, and requirements for protection of environmental or cultural resources. Although not comprehensive, the following considerations should be included in the levee construction contract documents.

10.7.2 Public and private utilities (natural gas, communications, water, sewer, cable television), railroads, highways (federal, state, and local), local business owners, and any other critical features impacted by the proposed levee construction should be identified during design including point of contact name and contact information. The construction contract documents should provide the timing of required notification to these stakeholders in advance of construction.

10.7.3 Existing utilities that require relocation or remain active during and after the levee construction activity should be identified and located. The presence of these utilities may interrupt and delay the completion of the exploration trench until they are relocated.

10.7.4 Available haul roads and points of entry into and egress from the construction site should be identified in the construction contract documents. The contractor may be required to make a video recording of the existing condition of all haul roads to be used during construction. The video will help determine the responsibility and extent of any repairs at the end of construction. The contractor may be required to clean haul roads at the end of each day of hauling, especially if the road is used to haul borrow material from an off-site borrow pit.

10.7.5 Daily working hours including weekends and appropriate work periods throughout the year should be specified in the construction contract documents. These limits should be determined based on consultation with the local sponsor and local environmental agencies, and should include consideration when working near residential or commercial areas and environmentally restrictive areas or sensitive habitats.

10.7.6 Sequence of construction, location, and data evaluation requirements for test fills or sections should be specified in the construction contract documents. The sequence of construction of test fills or sections will impact subsequent levee construction activities.

10.7.7 To prevent any bearing capacity issues, identify the maximum free-standing height of gravity drain gatewell construction (if applicable) above completed gravity drain pipe (if applicable) installation and earthwork occurring at the bottom of the gatewell.

10.7.8 <u>Third Party Agreements.</u>

10.7.8.1 Highlight any third party agreements that are part of the levee construction contract documents. A third party agreement is a legally binding, real estate agreement between USACE, the sponsor, and a major project stakeholder (tribes, environmental organizations, railroad, highway agency, utility, off-site borrow pit owner) whose existing infrastructure, cultural resources, or environmental resources will be impacted by the proposed levee construction activity. Examples of third party agreements are as follow:

10.7.8.2 A third party agreement might be executed with a railroad providing “no-train” windows on a main-line rail corridor to facilitate construction of a closure structure across that live track. The levee construction contractor must be made aware of the scope of his responsibilities to complete the work within the “no-train” windows.

10.7.8.3 A third party agreement might be executed with the owner of an active quarry which includes the location of an off-site borrow pit. The levee construction contractor must be made aware of any limitations placed upon him by the quarry owner in order to safely excavate, load, and haul borrow from the quarry.

10.7.8.4 A third party agreement might include instructions related to the timing of partial demobilization and protection of completed work should flood waters threaten the construction site.

10.7.8.5 A third party agreement might include limits on open excavations and requirements to close those excavations to limit impacts on adjacent businesses.

10.7.8.6 A third party agreement might contain requirements for on-site monitoring in environmentally or culturally sensitive areas during significant construction activities (i.e., excavations).

10.8 Levee Project Documentation and Post Construction Activities.

10.8.1 As-Built Drawings and Design Documentation Report. ER 1110-2-1150 specifies the requirements for as-built drawings and a design documentation report (DDR) for levee projects. Both the as-built drawings and DDR should be finalized when the levee project construction is complete. This documentation is very important for future assessors of the levee as it provides a fundamental level of information that will be compared to future levee performance and will inform levee rehabilitation activities. The DDR should contain design decisions made during construction and summary of construction issues and resolutions of construction issues for the levee project. In addition, the DDR should also contain information (measurements and courses of action) from implementation of the observational method during construction (Section 10.5.2.2). The DDR should also include as-built drawings, documentation of exploration trench conditions and observations, compaction reports and other construction testing results, and measurements taken during construction to verify design assumptions. The final DDR should be maintained with other pertinent documentation for the levee system and be readily assessable for future inspections and risk assessments.

10.8.2 Foundation and Embankment Report. Preparing a separate foundation and embankment report for the levee construction project is a good practice. A report should be prepared for levees that pose a significant threat to life safety and/or are fairly complex in nature (i.e., foundation conditions, embankment configuration, etc.). The foundation and/or embankment report for the levee should include a summary of foundation and embankment conditions, issues (and corresponding resolutions) encountered during construction, documentation of exploration trench conditions and observations, final foundation approval reports (if necessary), verification of design assumptions (i.e., shear strengths, hydraulic conductivity values, etc.), and records of construction testing.

10.8.3 National Levee Database. Post-construction levee information (e.g., levee alignment, leveed area, features details, etc) shall be included for new levees and/or updated for existing levees within the National Levee Database (NLD). The most current guidance for including and/or updating levee information within the NLD should be followed.

10.8.4 Post Construction Levee Risk Assessment. A post-construction levee risk assessment should be performed following the most current U.S. Army Corps of Engineers Levee Safety Program guidance.

CHAPTER 11

Special Features

Section I
Roads, Ramps, and Crossings

11.1 Access Roads.

11.1.1 Access Road to Levee. Access roads should be provided to levees at reasonably close intervals in cooperation with state and local authorities. The roads should be all-weather roads (e.g. gravel-paved, crushed rock-paved, asphalt concrete-paved, etc.) that will allow access for the purpose of inspection, maintenance, and flood-fighting operations.

11.1.2 Access Road on Levee. Access roads, sometimes referred to as patrol roads, should be provided on the levee crown and/or along either levee toe for the general purposes of inspection, maintenance, and flood-fighting operations. Access roads should also be provided for levee structures and appurtenances, including gate or closure structures, pump stations, and adjacent to relief wells. This type of road should be surfaced with suitable gravel or crushed stone base course that will permit vehicle access during wet weather without causing detrimental effects to the levee or presenting safety hazards to the levee inspection and maintenance personnel. Non-woven geotextiles or geogrids could 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. The width of the road surfacing will depend upon the crown width of the levee and whether it is meant to accommodate one- or two-way traffic. On levees where county or state highways will occupy the crown, the type of surfacing and surfacing width should be in accordance with applicable county or state standards. The decision as to whether the access road is to be opened to public use is to be made by the levee owner. The levee section should never be reduced to accommodate an access road. Both public and private roads should be constructed only by adding material to the levee crown and slopes.

11.1.3

11.1.3.1 Turnouts. Turnouts should be used to provide 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 and are particularly beneficial where there are no ramps within the reach. The exact locations of the turnouts will be dependent upon various factors such as sight distance, property lines, levee alignment, and desires of local interests. An example turnout for a levee with a 12-foot wide levee crown is shown in Figure 11-1.

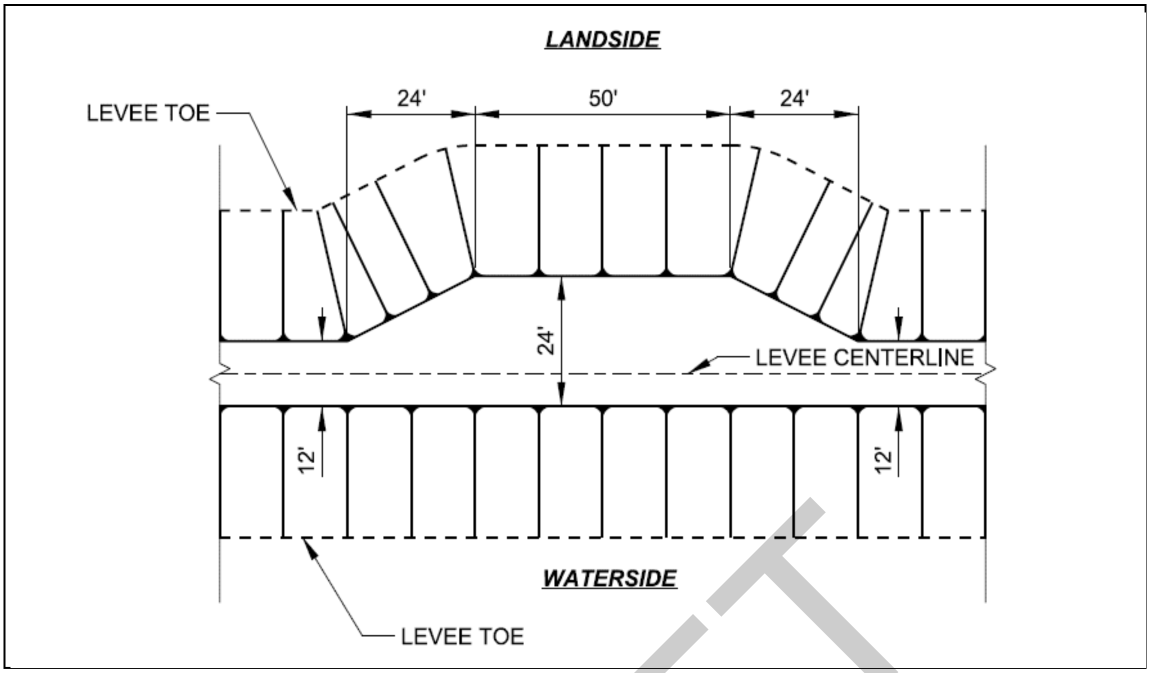

Figure 11-1. Example of Levee Turnout.

11.1.3.2 Turnarounds. Turnarounds are sometimes provided to allow heavy equipment to reverse their direction on levees. Turnarounds should be provided every couple miles and are particularly beneficial where no ramp exists near where the levee dead-ends. An example turnaround for a levee with a 12-foot wide crown is shown in Figure 11-2.

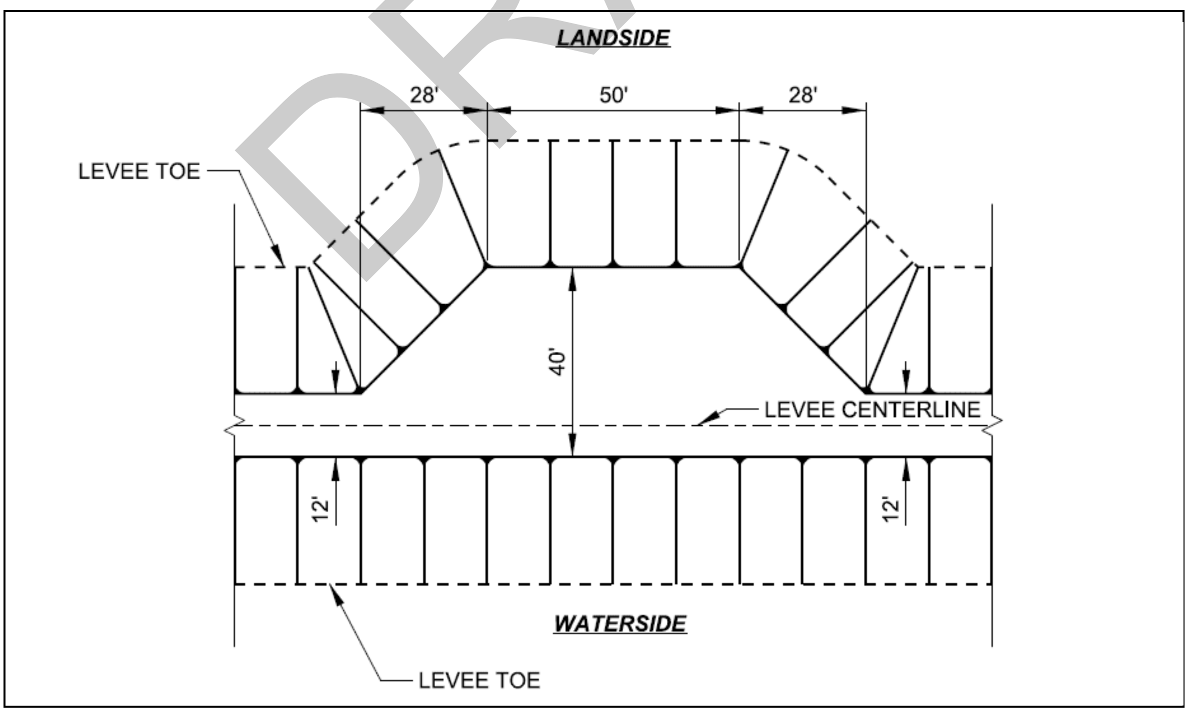

Figure 11-2. Example of Levee Turnaround.

11.2 Ramps.

11.2.1 Ramps should be provided approximately every mile to permit vehicular traffic to access onto and exit the levee crown. Ramps may be located on either the landside or the riverside of the levee. Ramps on the landside of the levee are provided to connect access roads on top of a levee with access roads leading to a levee and at other convenient locations to serve landowners who have property bordering the levee. Ramps are also provided on some occasions on the riverside of the levee to connect the access road on top of the levee with existing levee traverses where necessary. The actual locations of the ramps should have the approval of the levee owner. On the riverside of the levee, ramps should be oriented to minimize turbulence during high water. Designers should consult with the hydraulics team member to ensure that the riverside ramp is located where it will not cause any undesired consequences, such as induced flooding or erosion.

11.2.2 Ramps are classified as public or private in accordance with their function. Public ramps are designed to satisfy the requirements of the levee owner: state, county, township, or road district. Private ramps are usually designed with less stringent requirements and maximum economy in mind. Ramps should be angled for side-approach instead of at a right-angle (perpendicular to the levee access road) because less embankment material is needed with an angled ramp. The ramp width will depend upon its intended function. Some widening of the levee crown at its juncture with the ramp may be required to provide adequate turning radius. The grade of the ramp should be no steeper than 10 percent. Side slopes on the ramp should generally be the same slope ratio as the adjacent embankment slope and should not be steeper than 1 vertical on 3 horizontal (1V:3H) to allow grass-cutting equipment to operate. The ramp should be surfaced with suitable gravel or crushed stone. Consideration should be given to extending the gravel or crushed stone surfacing to the levee embankment to minimize erosion in any drainage feature between the ramp surface and the side slope of the levee. In general, private ramps should not be constructed unless they are essential and there is assurance that the ramps will be used. Unused ramps lead to maintenance neglect.

11.2.3 The levee section should never be reduced to accommodate a ramp. Both public and private ramps should be constructed only by adding material to the levee crown and slopes.

11.3 Incorporating Public Use Roadways in Levee Sections.

11.3.1 Roadways, Railroad Junctions, and Other Use Embankments Acting as Levees. Highway and railroad embankments or other non-levee features that act to exclude flood water will be considered to be part of a levee system for evaluation and design purposes. Embankments that function as levees also exist in water conveyance systems, navigation channels, recreation areas, and habitat restoration projects. These structures should not be incorporated into the levee system unless there is an agreement with the owner to allow access to the embankment and the features have been designed in accordance with USACE standards. Road, railroad, and other use embankments that serve or function as levees need to be continually operated and maintained to assure continued integrity.

11.3.2 The Federal Highway Administration (FHWA) published clear guidance to their field offices that FHWA does not have flood control standards for highway embankments and the existing highway system was not designed or intended to serve in a flood control role (FHWA 2008). Therefore, separate confirmation that a highway embankment incorporated in a levee system can also meet various levee design requirements is necessary.

Section II
Levee Enlargements

11.4 General. The term levee enlargement pertains to any addition to an existing levee that raises the grade and/or width. A higher levee grade, after a levee has been constructed, may be required for several reasons. Additional statistical information gathered from recent floodings or recent hurricanes may establish a higher project flood elevation on a river system or a higher elevation for protection from incoming tidal waves produced by hurricane forces in low-lying coastal areas. The most economical and practical approach to provide additional levee height is normally a levee enlargement. Levee enlargements are constructed either by adding additional earth fill or by constructing a flood-wall on the crown. The floodwalls could be I-walls or T-walls; however, because of poor performance, I-walls are now generally discouraged. Prior to any levee raise, a hydraulic evaluation of increased stages outside the leveed area would need to be performed to determine the hydraulic impact on others caused by raising the levee.

11.5 Earth-Levee Enlargement.

11.5.1 Earth-levee enlargement, when possible, is normally preferred to floodwalls since it is usually more economical and the wider embankment profile provides more resilience to many levee potential failure modes. This type of enlargement is used on both agricultural and urban levees where borrow sites exist nearby and sufficient right-of-way is available to accommodate a wider levee section.

11.5.2 An earth-levee enlargement is accomplished by one of three different methods: riverside, straddle, or landside enlargement. A riverside enlargement is accomplished by increasing the levee section at the crown and on the riverside of the levee as shown in Figure 11-3a. A straddle enlargement is accomplished by increasing the levee section on the riverside, at the crown, and on the landside of the levee as shown in Figure 11-3b. A landside enlargement is accomplished by increasing the levee section at the crown and on the landside of the levee as shown in Figure 11-3c. There are advantages and disadvantages to each enlargement method that will have to be considered for each project. The riverside and straddle enlargements would be more costly if the riverside slope has riprap/concrete protection and could also create hydraulic encroachment for narrow floodways that would impact top of levee designs and areas outside of levees. Landside enlargements would require additional right-of-way and larger fill quantities for levees with flatter landside slopes. Riverside enlargements often have more environmental impacts, may change the erosion pattern within the river, and require larger fill quantities for levees with either flatter riverside slopes or with a higher interior ground surface elevation. The straddle enlargement would require the whole levee system to be stripped with work being done on both sides of the levee (see Chapter 10 for a description of stripping).

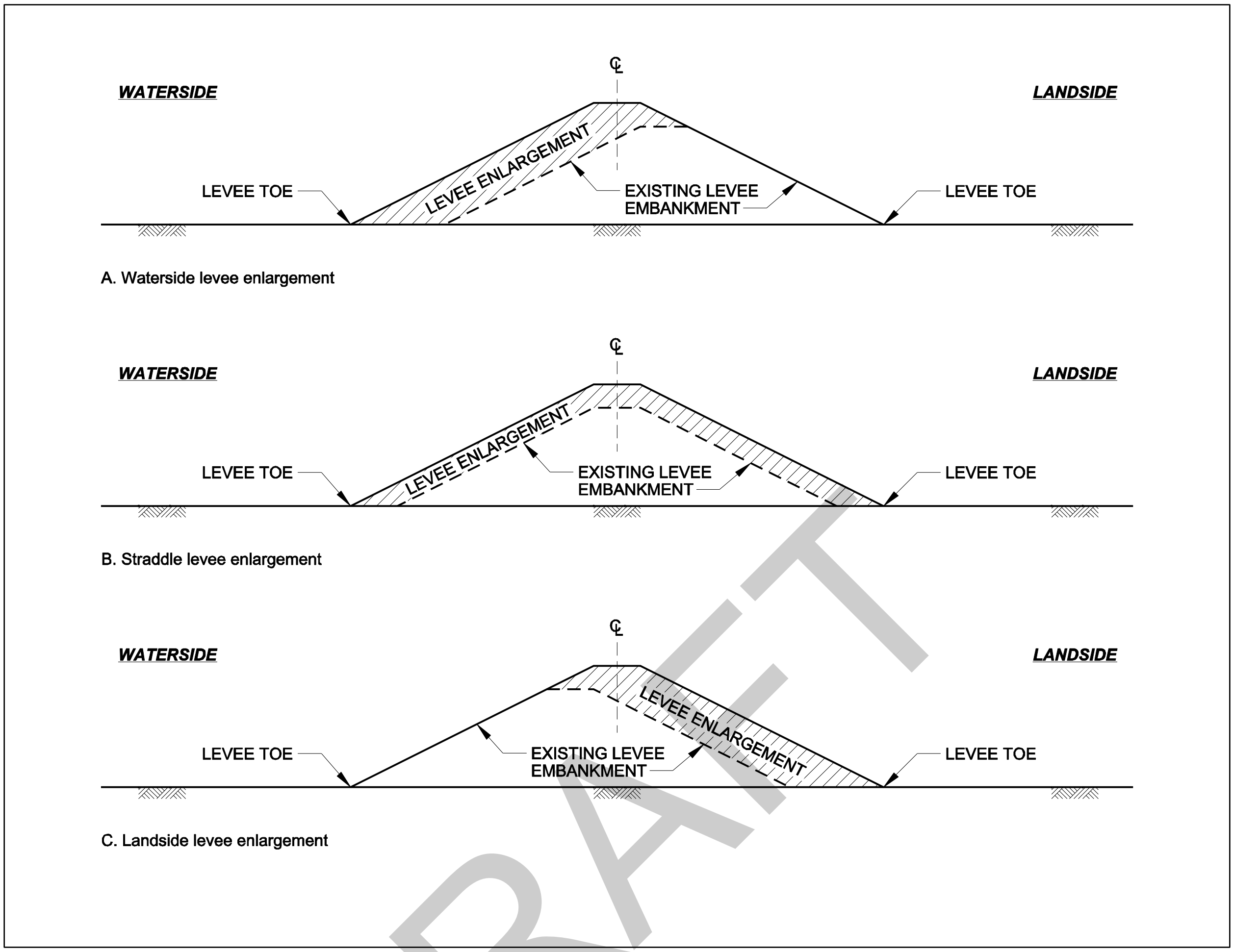

Figure 11-3. Enlargements.

11.5.3 The modified levee section should be checked for through-seepage and underseepage as discussed in Chapter 6, for foundation and embankment stability as discussed in Chapter 7, for settlement as discussed in Chapter 8, and for surface erosion as discussed in Chapter 9. Sufficient soil borings should be taken to assess the in-situ soil properties of the existing levee embankment and foundation for design purposes.

11.5.4 An earth-levee enlargement should be made integral with the existing levee. Every effort should be made such that the enlargement has at least the same degree of compaction as the existing levee on which it is constructed and may need to be compacted to meet minimum compaction standards over and above what the existing levee material is compacted to. Preparation of the interface along the existing levee surface and upon the foundation should be made to ensure good bond between the enlargement and the surfaces on which it rests. The foundation surface should be cleared, grubbed, and stripped to 15 feet beyond the final levee toe as described in Chapter 10. The existing levee surface upon which the levee enlargement is placed should also be stripped of all low-growing vegetation and organic topsoil. The topsoil that is removed should be stockpiled for reuse as topsoil for the enlargement. Exploration trenches may be necessary and can be excavated to ensure the new fill will be placed on competent, stable foundation material. Prior to constructing the enlargement, the stripped surfaces of the foundation and existing levee should be scarified before the first lifts of the enlargements are placed. Horizontal lifts should be benched into the existing embankment at least 1 to 2 feet as described in Section 10.3.4.2. The widths of the additional fills should be such that normal-width construction equipment (8 feet wide) can be used for compaction. Sliver fills, narrow fill widths of about to 2 to 4 feet, should rarely be allowed. Additional care must be taken with landside levee enlargement because any weakness (e.g., cracking or separation) between the existing levee section and the levee enlargement at the level of the original crown is subject to seepage infiltration during high water. There is an opportunity to improve levee integrity with a waterside levee raise if the levee has a through-levee seepage problem. Placement of an impervious blanket on the waterside during a waterside levee raise could reduce through-levee seepage. It should also be noted that, on the other hand, the placement of a semi-pervious or impervious blanket on the landside slope during a landside raise could reduce levee integrity and stability if the existing levee is relatively pervious. If it is not practical to use fill for landside enlargements that is more permeable than the existing embankment fill, either: 1) a drainage layer should be provided at the interface between the existing embankment and the new fill that drains to the landside toe or 2) the landside enlargement width should be oversized to achieve a stable configuration.

11.6 <u>Floodwall-Levee Enlargement</u>.

11.6.1 A floodwall-levee enlargement is used when additional right-of-way is not available or is too expensive, or if the foundation conditions will not permit an increase in the levee section. Economic justification of floodwall-levee enlargement cannot usually be attained except in urban areas. Two common types of floodwalls that are used to raise levee grades are the I-wall and the inverted T-wall. Although less common, pile-founded T-walls have successfully been used where near-surface soils are weak and/or the wall is subject to high loads. The design of pile-founded T-walls typically requires advanced soil structure interaction (SSI) analyses and are unique structures that go beyond the scope of this manual.

11.6.2 An I-wall is a cantilever vertical wall partially embedded in the levee crown. The stability of such walls depends upon the development of passive resistance from the soil. ECB 2017-3 transmits specific guidance to be used for the design and evaluation of I-walls and sheet pile walls. I-walls require a global evaluation of the wall-levee system because analysis methods used for cantilever walls assume a flat ground surface. One common method of constructing an I-wall is by combining sheet pile with a concrete cap as shown in Figure 11-4. The lower part of the wall consists of a row of steel sheet piles that are driven into the levee embankment, and the upper part is a reinforced concrete section capping the steel piling. For stability reasons highlighted by poor performance of these walls during Hurricane Katrina that were documented in the Interagency Performance Evaluation Task Force (IPET 2007), I-walls are generally discouraged, particularly for heights greater than 6 feet, and gravity walls or reinforced concrete walls, such as T-walls, are much preferred.

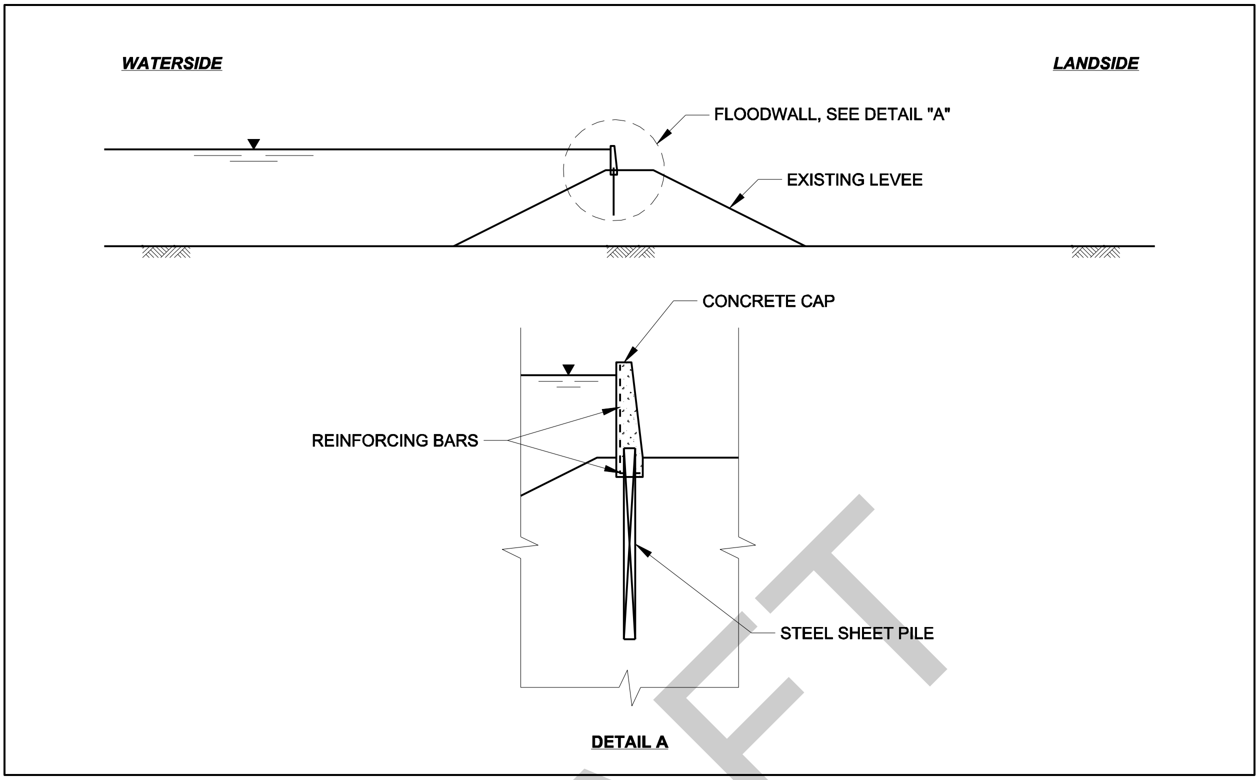

Figure 11-4. I-type Floodwall-Levee Enlargement.

11.6.3 An inverted T-wall is a reinforced concrete wall whose members act as wide cantilever beams in resisting hydrostatic pressures acting against the wall. A typical wall of this type is shown in Figure 11-5. The inverted T-wall is used to make floodwall levee enlargements when walls higher than 6 feet are required.

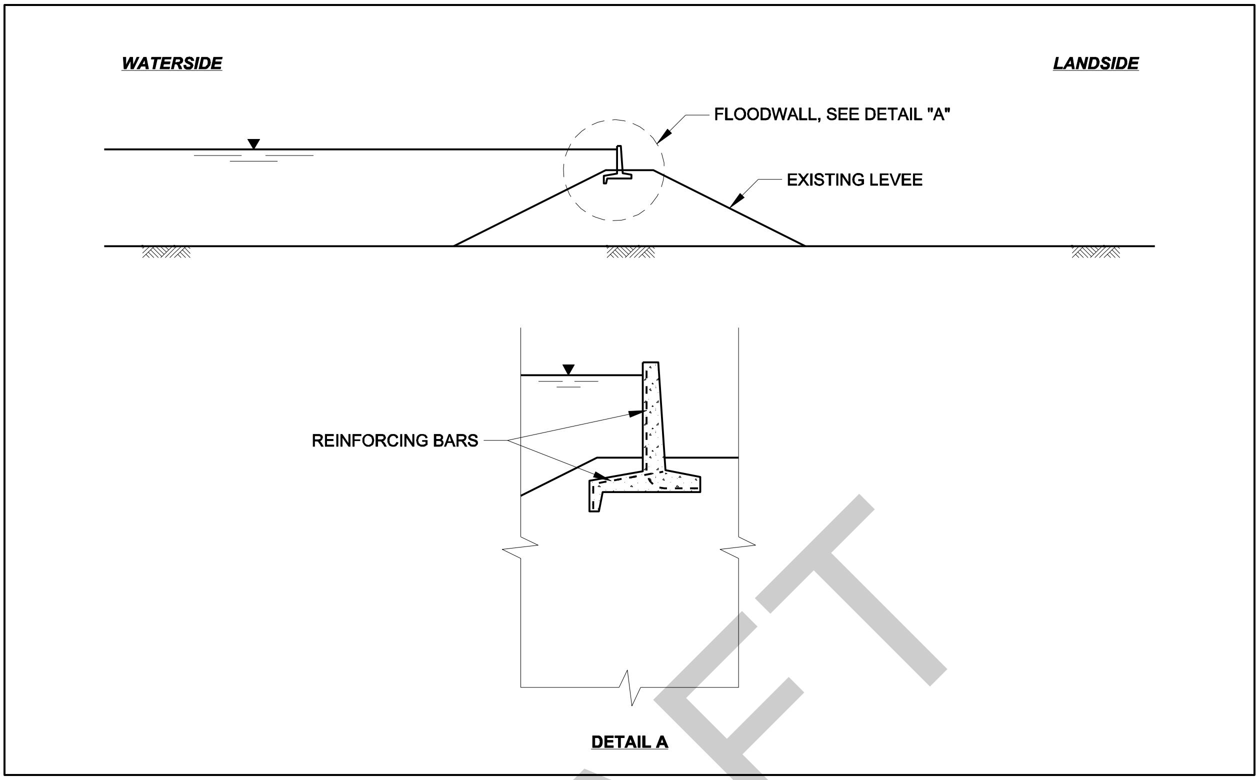

Figure 11-5. Inverted T-type Floodwall Levee Enlargement.

11.6.4 The floodwall should have adequate stability to resist all forces which may act upon it. An I-wall is considered stable if sufficient passive earth resistance can be developed for a given penetration of the wall into the levee to yield an ample factor of safety against overturning. The evaluation or design of I-walls founded on levees should consider the technical concerns described in Duncan et al. (2008). The depth of sheet-pile penetration of the I-wall should be such that adequate seepage control is provided. Normally the penetration depth of the I-wall required for stability is sufficient to satisfy the seepage requirements, but if wall deflection results in a potential gap on the riverside of the sheeting, full hydrostatic pressure in that gap should be considered. For the inverted T-wall, the wall should have overall dimensions to satisfy the stability criteria and seepage control as presented in EM 1110-2-2502 and EM 1110-2-2100.

11.6.5 The existing levee section should be checked for through-seepage and underseepage as discussed in Chapter 6 and for embankment and foundation stability as discussed in Chapter 7 under the additional hydrostatic forces expected. If unsafe seepage forces or inadequate embankment stability will result from the higher heads, seepage control methods as described in Chapter 6 and methods of improving embankment stability as described in Chapter 7 may be used. However, some of these methods of controlling seepage and improving embankment stability may require additional right-of-way for construction which could eliminate the economic advantages of the floodwall in comparison with an earth-levee enlargement. As in earth-levee enlargements, a sufficient number of soil borings should be taken to assess the in-situ soil properties of the existing levee embankment and foundation for design purposes.

11.7 Mechanically Stabilized Earth (MSE) Structures-Levee Enlargement/Reduction.

11.7.1 MSE walls as not typically used in levee systems and requires approval by CECW-EC for their use within a levee system. MSE walls have been used with levees to reduce the footprint of the levee section. This tends to shorten seepage paths and potentially reduce levee stability. However, with a facing material that requires minimal maintenance, MSE walls may be incorporated into levee embankments provided they are properly designed and constructed. The levee/MSE wall section must meet all controlling criteria including levee criteria as presented in this manual and MSE wall criteria. The section must also be designed for dissimilar materials considering filtration and settlement. Often MSE walls are constructed using reinforced backfill materials that are granular to promote drainage behind the wall-facing. These materials may differ from levee fill and may provide little head reduction if constructed on the waterside of a clay levee or may act as a drainage medium if constructed on the landside. There are many MSE wall systems on the market so it is difficult to complete a generic MSE wall design for bidding purposes that is efficient with respect to reinforcement spacing (internal stability). External stability (sliding, overturning, bearing capacity) and global stability typically control the reinforcement length and can be reasonably designed by the levee designer, not the MSE wall designer (if different).

11.7.2 MSE slopes, typically referred to as reinforced steepened slopes, are often constructed with vegetated slopes that cannot be maintained using standard mowing equipment. Proper vegetation maintenance should be determined and included in the O&M manual where these products are used in conjunction with levees.

Section III
Junction with Concrete Closure Structures

11.8 General. In some areas, a flood protection system may be composed of levees, floodwalls, and drainage control structures (gated structures, pumping stations, etc.). In such a system, a closure must be made between the levee and the concrete structure to complete the flood protection. One closure situation occurs when the levee ties into a concrete floodwall or a cutoff wall. In this closure situation, the wall itself is usually embedded in the levee embankment. In EM 1110-2-2502, a method of making a junction between a concrete floodwall and levee is discussed and illustrated. Another closure situation occurs when the levee ties into a drainage control structure by abutting directly against the structure as shown in Figure 11-6. In this situation, the exterior of the abutting end walls of the concrete structure should be battered at an angle of 10V:1H to ensure adequate compaction and a firm contact between the structure and the fill. In addition to the battered concrete walls, concrete wingwalls and sheetpiles are often used to extend beyond the concrete structure and into the earthen sections along the levee centerline.

11.9 Design Considerations. When joining a levee embankment with a concrete structure, concerns that should be considered in the design of the junction are differential settlement, compaction, seepage, and embankment slope protection.

11.9.1 Differential Settlement. Differential settlement caused by unequal consolidation of the foundation soil at the junction between a relatively heavy levee embankment and a relatively light concrete closure structure can be serious if foundation conditions are poor and the juncture is improperly designed. Preloading has been used successfully to minimize differential settlements at these locations. In EM 1110-2-2502, a transitioning procedure for a junction between a levee embankment and a floodwall is presented that minimizes the effect of differential settlement.

11.9.2 Compaction. Thorough compaction of the levee embankment at the junction of the concrete structure and levee is essential. Good compaction decreases the hydraulic conductivity of the embankment material and ensures a firm contact with the structure. Heavy compaction equipment such as pneumatic or sheepsfoot rollers should be used where possible. When used in the immediate vicinity of vertical non-yielding walls, heavy compaction equipment can “lock-in” high residual lateral stresses against the wall as described in ECB 2017-2. In confined areas such as those immediately adjacent to concrete walls, compaction should be by hand tampers in thin loose lifts as described in EM 1110-2-1911.

11.9.3 Seepage. Seepage needs to be analyzed to assess the necessary embedment length of the structure-levee junction. Zoning of the embankment materials, as described in Section 10.3, needs to be maintained through the junction unless analysis indicates different zoning is required. A properly designed landside filter at the interface between the embankment and structure will reduce the risk of internal erosion of the embankment material through cracks that can develop along the interface.

11.9.4 Slope Protection. Slope protection should be considered for the levee embankment at all junctions of levees with concrete closure structures. Turbulence may result at the junction due to changes in the geometry between the levee and the structure. This turbulence will cause scouring of the levee embankment if slope protection is not provided. Slope protection for areas where scouring is anticipated is discussed in Chapter 9.

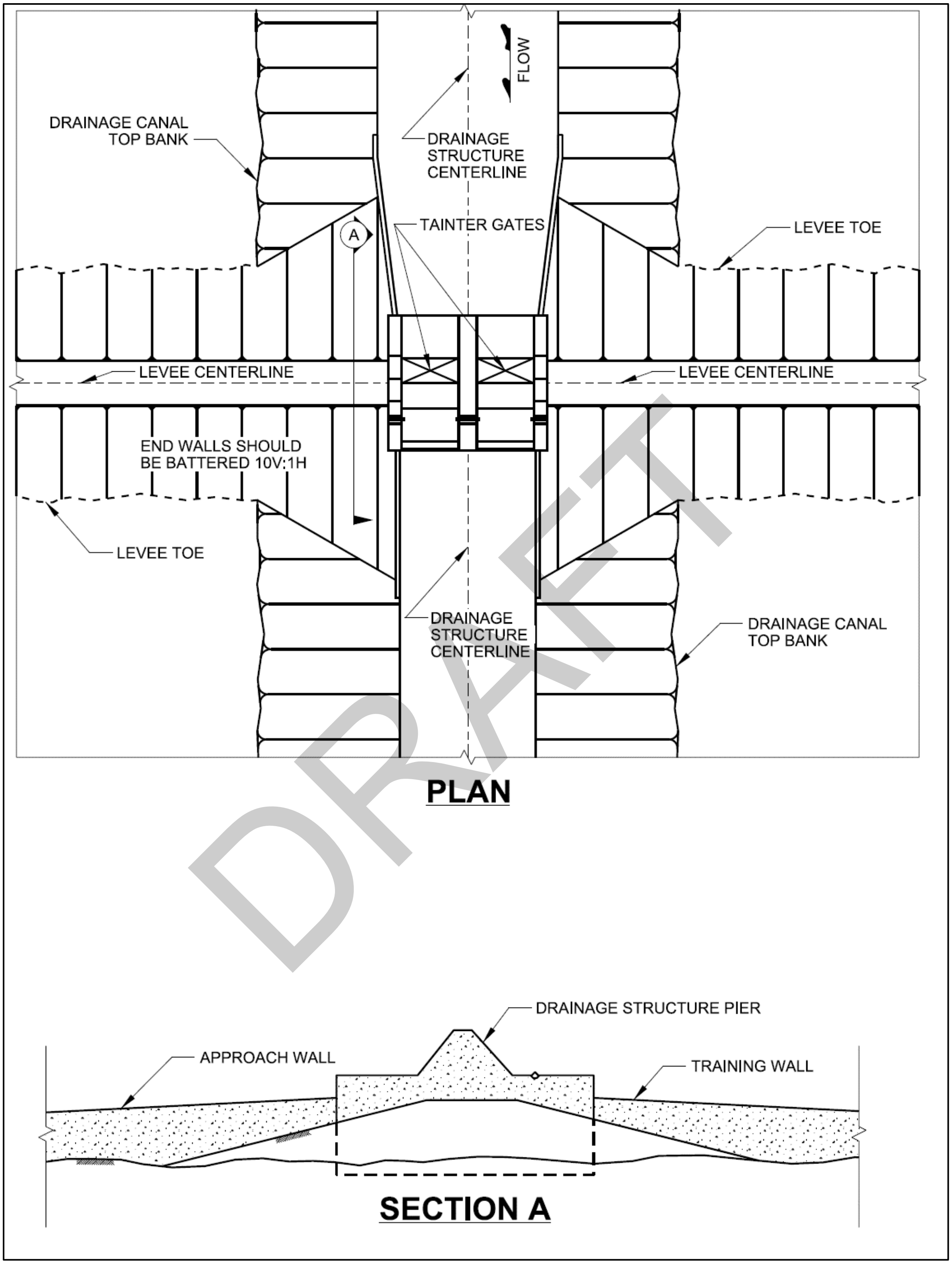

Figure 11-6. Junction of Levee and Drainage Structures.

Section IV
Vegetation and Encroachments

11.10 Vegetation.

11.10.1 The proper consideration of vegetation in levee design and management of vegetation after construction is critical to levee performance, especially within the footprint of the levee and adjacent levee corridors described in Section 1.6.1.4. Prior to levee construction this area needs to be cleared and grubbed as described in Chapter 10 for new levees and Section 11.5 for levee enlargements. Best practice is to keep the vegetation-free zone described in Chapter 1 free from woody growth for the life of the project.

11.10.2 Because landscape planting, or vegetation, enhances the environment, with respect to both natural systems and human use, it is to be considered in all flood-risk-management-infrastructure project planning and design studies and should be fully discussed in design documentation reports.

11.10.3 The integrated design of landscape plantings and vegetation management at flood-risk-management-infrastructure requires a coordinated, interdisciplinary effort involving the local sponsor and the following disciplines: civil engineer, landscape architect, levee and/or dam safety engineer, environmental engineer, geologist, biologist, and additional related disciplines, as appropriate.

11.10.4 Vegetation can only be incorporated into a levee system to provide environmental and aesthetic benefits when the required reliability of flood-risk-management infrastructure is not compromised A key component of this guidance is the prescription of a minimum vegetation-free zone as prescribed in Section 1.6.1.4. The use of vegetation zones smaller than those recommended in Section 1.6.1.4 can be incorporated into a levee design provided it is evaluated with a risk assessment, proper operations and maintenance practices are developed for the allowed vegetation, and there is confidence that sufficient funding will be available over the life of the project to maintain the vegetation.

11.10.5 Beyond the Vegetation Free Zone prescribed in Section 1.6.1.4, assessing or anticipating vegetation growth adjacent to levees, and its potential to affect performance with respect to stability and seepage, is a very complex issue and should be evaluated using Potential Failure Modes Analysis methods described in Chapter 1. Each situation is unique, and there is no prescriptive process for the conduct of such an assessment. Each situation requires site-specific consideration by experienced, knowledgeable personnel. Important factors to consider include the proximity of the vegetation growth to the levee, the density of the vegetation growth, type of vegetation, past performance of the levee under significant load, geological and geotechnical properties of the levee and its foundation, construction methods, duration of head pressures significant enough to initiate seepage and/or piping, flow velocities, and the ability to detect issues as they arise during a flood event. Larger trees will tend to be of greater concern than smaller trees. Dense vegetation is generally of greater concern than sparse vegetation, except where isolated trees may be more likely to induce scour. Conversely, an expanse of dense vegetation, waterward of the levee, may reduce impact from wind-generated waves. Potential impacts of tree overturning – driven by wind or other loading, such as ice or snow – must also be considered. Large trees with extensive and/or deep root systems, when overturned and uprooted, can remove a significant soil mass, potentially impacting global stability and seepage resistance: where this occurs on the waterside of the levee, eddy scour conditions may develop and may induce lateral erosion into the bank and/or levee embankment.

11.11 Encroachments. An encroachment is a third-party, non-project feature that exists inside the boundaries of the federal project. All existing encroachments within the project footprint should be evaluated to determine if they will negatively impact the project. Encroachments should not impact structural integrity of the levee, hydraulic conveyance of the channel and floodway, operation, maintenance, or flood fighting activities. Any encroachment that is determined to impact that project should be removed/altered/or relocated, as necessary. For projects locally maintained, USACE should coordinate this effort with the local sponsor, and work with them to assign and/or update permits of encroachments, as necessary. All encroachments should be properly permitted.

11.12 Miscellaneous Penetrations. Pipeline crossings are discussed in EM 1110-2-2902. Other penetrations such as bridge piers, telephone poles, etc. must not impact structural integrity of the levee, hydraulic conveyance of the channel and floodway, operation, maintenance, or flood fighting activities. All encroachments should be identified (location, coordinates, owner, date installed, alteration approvals or modifications, most recent condition, and description of the nature of the penetration) and their potential impacts to levee integrity or performance should be assessed. This information should be maintained in a database and updated periodically by inspections.

Section V
Regional Impacts on Design

11.13 Seismic Considerations. As stated in Chapter 7, levee seismic performance is generally not of concern because for many levee systems the probability of having a flood during or shortly after an earthquake but before post-seismic damage repairs have been made is very low. However, it is recognized that many levees are susceptible to damage during seismic events and any loss of levee height or damage to critical features may need to be repaired before the next flood event, especially if life loss or economic damages would be high. Qualitative and quantitative risk assessments that consider frequency of flooding, estimated ground motions, levee profile, levee and foundation soil strength, and consequences of inundation are helpful for evaluating whether further actions are required. Further actions to consider include additional site investigations, detailed seismic analyses, remedial designs, enhanced emergency response procedures, and land-use policy decisions. Detailed seismic evaluations should assess the impacts to levee performance as well as seepage control measures such as relief wells, seepage cutoff walls, and toe drains. Selection of an appropriate seepage/stability reinforcement alternative should include the potential for seismic damage to the feature. Damage to these features could result in levee failure. Seepage control measures are typically located along the landside levee toe and are especially susceptible to damage from even relatively minor shear deformations and differential settlements. If ground improvement is not economical and the levee does not impound flood flows on a nearly continuous basis, it may be prudent for the local district to identify in advance potential borrow sources for restoration of levee height/profile.

11.14 Extreme Climate Considerations. Regional climate differences are reflected in different levee design and construction by region; differences to consider include wet/dry, hot/cold drought, frost, etc. Desiccation cracking can result in a shortened seepage path through an otherwise low permeability layer. Dry, arid climates are not suitable for sod commonly used on levees for erosion protection, and many of these dry, arid regions are also subject to flashy streams with high velocity, making them especially vulnerable to erosion. Regional subsidence could lead to cracking in a seepage cutoff. Frost heave can damage seepage control features and provide a shortened seepage path along the base of a structure. Drainage features can be compromised when ice forms in discharge pipes. The freezing and thawing of ice in cracks can cause rip rap to deteriorate and reduce the erosion protection over time. The design Potential Failure Modes Analysis should consider the potential impacts of the project extreme climate conditions on expected performance and design components and address potential inadequacies.

11.15 Levee Breach Repair Considerations. The design and construction of levee breach repairs, including those repairs completed under PL 84-99 authority, should be completed using the criteria presented in this engineering manual. The cause of the levee breach (e.g., whether the levee breached as a result of overtopping or as a result of breach prior to overtopping failure mode) should be determined, thoroughly evaluated, and be considered during design and construction of the breach repair. The scour of the levee and foundation that occurs during a levee breach can result in significantly changed site conditions. Site conditions along the alignment of the repair levee section should be thoroughly re-evaluated and interpreted as described in Chapters 2 and 5 with special consideration of the impacts of any scour that occurred during breach. Changes to the levee alignment to the repair the breach should consider any hydraulic impacts that have negative impacts to water surface profiles upstream or downstream and that could increase river velocities and scour along the levee.

Section VI
Closure Structures

11.16 Closure Structure Decision Criteria. The decision of when a closure structure is needed and where to place it along the levee system should be carefully considered by planners and engineers to ensure the best possible design for the full life cycle of a project. Without a proper design, construction, operation, and maintenance of a closure structure, these components of the system can introduce additional risk that may not have been considered or accounted for in the planning and engineering phases of a project. The following sections outline the major considerations for planners and engineers to consider when a closure structure is believed to be required for a project.

11.16.1 Selection of Closure Structure Location. There are no simple rules to help designers select a location for a closure. The planning and design teams should coordinate with the local sponsor to determine where closures should be placed to accommodate both levee safety and public access needs. This may consist of raising roads to match the final design grade, abandoning/rerouting roads, installation of hardware for gate or panel closures, or determination that closure will be made with sandbags. Designers should understand that if an existing street no longer connects to an intersecting street because a levee has been constructed across its alignment there may be an adverse impact to evacuation routes in the event of a flood. This should also be taken into consideration when determining the location of a closure structure.

11.16.2 Cost of Design, Construction, Storage Requirements, Operation, and Maintenance. The cost of design and construction is usually the primary focus of any project. However, the operation and maintenance costs of some closures may not be fully appreciated or understood by designers; this could result in higher maintenance expenses over the levee system's service life than the local sponsor is willing or able to bear. Consideration should be given to lower-cost alternatives (e.g., sandbag closure), but designers should understand that lower-cost alternatives can be impractical in some emergency operations and higher-cost alternatives may be required. Repair of deteriorated concrete and embedded steel components, practice and training installations, availability of future replacement parts, storage, and manpower requirements are all costs that should be considered by planners and engineers 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. Additionally, Operation and Maintenance Manuals should include all facets of the selected closure structures, including parts' diagrams and lubrication points.

11.16.3 Selection of Closure Type. The type of closure to be used in a project is an important decision, as it can have far reaching implications. There are many different scenarios in which a closure is required and how a design team selects the type of closure can be based on many different factors, which cannot all be covered in guidance. However, designers should consider how different closure types may provide benefits such as resiliency and efficiency, while others may be costly or time intensive to operate. For example, when a closure is needed across a road or a railroad track, moveable gates are probably 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 especially important decision factor for active railroad lines. This type of closure also requires significantly fewer personnel to install than a typical stoplog closure. Closures that may require large equipment, such as cranes, for installation should be avoided when possible. Rolling gates of a wide variety of lengths and heights can be designed where real estate is available to store the gate while not in operation. Swing gates have been successfully employed on many projects; however, they are more limited in the length of span they can close, and similar real estate limitations as discussed for rolling gates apply. Conversely, installation of either a stoplog or sandbag closure in the same location would block traffic over a longer duration of time prior to the arrival of floodwaters. Neither of these types of closures can be easily "reopened" if needed. Additionally, the relative rise and fall of the applicable floodwater source is an important consideration. Levee systems that provide risk reduction against flashy water sources (those that rise and fall within a matter of hours) should only utilize closure systems that can quickly be closed as there will be insufficient time to set certain types of closures (sandbags, soil piles, stoplog systems, etc.). Finally, sandbag closures should be limited to no taller than four feet.

11.16.4 Operational Equipment, Time, and Manpower Requirements. The equipment, time, and manpower required to operate an individual closure should be carefully considered and documented during the design phase of the project. Additionally, designers should consider how an individual closure may be operated as part of a larger system. In many cases, a series of closures may be required as part of a single levee system. In these instances, designers should ensure the required time and manpower to operate all closures is carefully considered and this should be coordinated with future sponsors/owners/operators of the levee to ensure sufficient manpower and equipment resources will be available so all closures in a system can be properly placed in a timely manner. If it is believed that not all of a system's closures could be successfully placed in the lead up time before an event, the design of some closures may need to be reconsidered. Alternately, some closures may need to be eliminated.

11.16.5 Alignment of Closures. When possible the levee alignment should be perpendicular to the roadway or railroad where any closure is required. This will result in minimizing the width of the closure opening, which should lower not only the construction cost of the closure, but also shorten the installation time.

11.16.6 Although not all closure considerations can be accounted for in guidance and these designs can be complex, planners and engineers should ensure that all components of closures are carefully thought out and accounted for in design. This will ensure a thorough and complete design to help manage risk of a project throughout the entire life cycle. The recommend closure types outlined in Table 11-1 should be used as a guide to assist designers with closure considerations. The use of closure systems different than recommended in Table 11-1 should only be used in conjunction with a project-specific operational equipment, time, and manpower analysis completed to support an evaluation or design risk assessment.

Table 11-1. Recommended Closure Types for Different Design Scenarios

Hydraulic Hazard Condition	Closure Location or Height	Closure Length	Recommended Type of Closure Structure
"Flashy" Stream or River	Roadway[1] or Railroad[2]	Any	Swing Gate, Rolling Gate, or Trolley Gate
"Slow Rising" Stream or River	Closure Height > 4 feet[3]	Any	Swing Gate, Rolling Gate, Trolley Gate, or Stoplog[4]
	Closure Height < 4 feet	< 100 linear feet[5]	Swing Gate, Rolling Gate, Trolley Gate, Stoplog, Soil Pile or Sand Bags
		> 100 linear feet[5]	Swing Gate, Rolling Gate, Trolley Gate, Stoplog, Soil Pile or Soil Basket
Coastal Storm Risk Management Systems[6]	Navigation Channel with Reverse Loading	Any	Sector Gate
	Navigation Channel only Loaded from One Side	Any	Sector Gate or Vertical Lift Gate
	Structures on Land	Any	Swing Gate, Rolling Gate, or Trolley Gate

[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 stoplog closures take a much larger contingent of manpower to install compared to swing gates or rolling gates. A decision to use this type of closure must be based on an understanding of local manpower resources available during a flood emergency.

[5] Using length of closure for a decision must be based on an understanding of local manpower resources available during a flood emergency.

CHAPTER 12

Development of Levee Operations and Maintenance Manual

12.1 Purpose.

12.1.1 All levee projects require maintenance and operation of the levee and appurtenant features. When projects are first constructed, they meet minimum specified levels of design. As levees age, there are design aspects that improve, such as better establishment of shallow-rooted grasses or consolidation and strengthening of soft foundation clays. There are also detrimental factors, such as cumulative damage from backwards erosion piping, reduced relief well efficiency, burrowing or rooting animals, extensive root penetrations, or human encroachments, that tend to reduce project reliability. To prevent loss of reliability and degradation of expected performance, projects must be routinely maintained and structural/mechanical features must be exercised and maintained to help ensure the projects perform as intended/designed.

12.1.2 The lead engineer/designer is responsible for including the project's expected needs and requirements in the operations and maintenance (O&M) manual. It is important to understand that proper inspection and maintenance are critical to ensure levee systems continue to perform after construction is complete. Lead engineers/designers also need to understand and appreciate the operation and maintenance effort in order to design the project in a way that improves the ability to efficiently and economically operate and maintain it. Projects are often expected to last longer than the 50-year economic life span typically used in the planning process. Projects that are difficult to operate and maintain run the risk of being operated improperly or not being maintained to expected levels, and could deteriorate before the end of the 50-year economic life span, whereas a properly maintained project may exhibit satisfactory performance for longer than 50 years.

12.2 Responsibility.

12.2.1 The District Commander is responsible for the operations, maintenance, repair, replacement, and rehabilitation manuals (OMRR&R manuals) in accordance with ER 1110-2-401. Typically, the district's Engineering Division is tasked with preparing the O&M Manuals.

12.2.2 For projects transferred to a local sponsor, the local sponsor has the sole responsibility for operation, maintenance, repair, replacement, and rehabilitation. The project delivery team (PDT) should consider the effort and costs associated with OMRR&R when designing the levee. Increased OMRR&R costs should not be used in place of good engineering practices or reduction of upfront design and construction costs. ER 415-1-11 specifies an O&M Plan be developed during design and an operability review during Biddability, Constructability, Operability, Environmental, and Sustainability (BCOES) reviews.

Section I
Operations and Maintenance Issues to Consider During Design

12.3 Encroachments. Encroachments of any type are not allowed within the project right of way unless approved. If encroachments are needed, they shall be evaluated by the PDT and approved by the Levee Safety Officer. An encroachment is defined as any physical object that can affect the integrity of the levee, limit access to or limit visual inspection of the project. Examples of encroachments include utility poles, sheds, swing sets, retaining walls, fences, stairs, trampolines, etc. The design of the levee should eliminate, or at least minimize, the need to approve encroachments. Approved encroachments shall be documented in the design reports.

12.4 Conduits, Culverts, and Pipes. Conduits, culverts, and pipes are required to be visually inspected or tested routinely as part of the Levee Safety Program. Video inspection can be performed for pipe penetrations. When designing the conduits, culverts, and pipes, the PDT must consider how these features will be inspected and/or tested in the future. Additional manholes and cleanouts may be required to facilitate inspections and/or testing. More information about culverts and other pipe penetrations through levees is provided in EM 1110-2-2902.

12.5 Vegetation.

12.5.1 Vegetation cover is a very important for levees. The PDT will select appropriate species of vegetation that will provide for good ground cover, especially if needed for erosion mitigation. In selecting species, the PDT will also consider what means are available for maintaining the vegetation (for example, mowing or burning). Species that thrive when burned yearly should be avoided if burning is not allowed.

12.5.2 Refer to current guidance to assure that vegetation provides environmental and aesthetic benefits without compromising the reliability of flood risk management infrastructure. Vegetation on levees should only be incorporated into a levee system when evaluated through a risk-informed design process (Chapter 1) and properly accounted in O&M activities including costs.

12.6 Animals and Rodents. Animals and rodents can cause significant harm and reduce the structural integrity of the levee. Animals such as cattle traversing the levee can cause a loss of vegetation cover, increasing the risk of levee erosion. Tunnels of burrowing animals will decrease the seepage path length and reduce the integrity of the levee.

12.6.1 Prevention Measures. The PDT should consider the different types of animals that could be present at the project site. The PDT should investigate preventive measures that can be incorporated into the design to reduce and eliminate animal and rodent issues. The prevention measures may depend on types of infestation, environment, land use, and embankment and foundation conditions.

12.6.2 Control. The PDT should be aware of the applicable laws and regulations associated with controlling the expected animals on the project.

12.6.3 Repair of Animal Burrows. Animal burrows must be repaired as soon as possible after discovery. At least two different methods can be implemented when repairing burrow holes. The first method is to fill the burrow with an earth, cement, water slurry through a pipe placed in the burrow. Care must be taken that the pipe does not extend too high above the ground surface, which could cause hydrofracturing of the levee. This method will require an engineer to design the grouting procedure in accordance to ER 1110-1-1807. The second method is to excavate the burrow area and then backfill the area with similar type material compacted to the requirements of embankment fill with considerations that the repair area does not create a localized plug condition. Care must be taken that the excavation does not affect the integrity of the levee and that the entire burrow is excavated. The backfill must be completed in a timely manner and as soon after excavation as possible.

12.7 Instrumentation. Chapter 2 includes discussions on groundwater monitoring and Chapter 8 includes discussions on settlement monitoring. Piezometers and settlement monitoring measures can be designed in accordance with EM 1110-2-1908. Ability to read monitoring instruments during high water events should be considered in design.

12.8 Malfunction of Levee System Components Potential Failure Modes.

12.8.1 Levee system components include closure systems and interior drainage features such as drainage structures or pumping stations. The failure of these components may be due to improper operation, installation, structural failure, mechanical failure, or any of the potential failure modes previously discussed (internal erosion, stability, etc.). Improper operation of a levee system component is generally due to human factors that may include not installing a closure system or not closing a drainage structure gate prior to a flood event. Loss of electrical power to a levee system component can also cause the component to not operate properly or at all. Structural or mechanical failure of a levee system component may be due to lack of proper operation and maintenance or excessive age of the levee system component (among other causes). Depending on the risk associated with the malfunction of the levee system component, redundancies in the mechanical and structural features may be necessary. Factors that affect malfunction of levee system components include:

- Operation and maintenance diligence
- Operational complexity of the levee system components
- Age of the levee system components
- Structural and mechanical resiliency and redundancy of the levee system component
- Redundancy in electrical power (if required) of the levee system component

12.8.2 When considering malfunction of levee system components potential failure modes, flow of flood waters through levee component may be limited due to the cross sectional area of the levee component. If the levee component cross sectional area is rather small (as is the case for a small culvert or levee system closure) full inundation of the leveed area may not occur due to the reduced hydraulic capacity through the levee component opening and duration of the flood event.

Section II
Operations and Maintenance Issues to Consider for the Project Life

12.9 Inspections and Risk Assessments. All levees are required to be inspected as part of the flood risk management system. Risk assessments should also be updated periodically. Engineering Circular (EC) 1165-2-218, Levee Safety Policy and Procedures, defines the requirements for inspections and risk assessments.

12.9.1 Local Sponsor Inspections. The local sponsor should routinely inspect the project throughout the year. Inspections should be conducted before the start of the flood season and after any major high water.

12.10 Routine Monitoring. The local sponsor will be responsible for routine monitoring of identified areas of concern. These areas of concern may be observed during local sponsor inspections and USACE inspections.

12.11 Expectations for Flood Fighting. The local sponsor will be responsible for flood fighting during an event in which water is on the levee. The local sponsor should prepare for flood events and be ready to conduct inspection of the project during events. The local sponsor will be responsible for conducting on-going surveillance activities during flood events and undertake flood fighting activities, as necessary.

Section III
Topics to Include in Operations and Maintenance Manual

12.12 General. The PDT will generally be tasked to prepare the O&M Manuals. The PDT will review ER 1110-2-401 for instructions on how to prepare the O&M Manuals. There are important topics to include in the O&M Manuals, including but not limited to those listed below.

12.13 Alterations. Alterations to the project without USACE permission is not allowed. Requestors of alterations to the levee project are required to follow the requirements of EC 1165-2-220, often referred to as the "Section 408 permitting process" when requesting alternations to any USACE federally authorized civil works project.

12.14 Surveys.

12.14.1 The top of levee elevations should be periodically surveyed and compared against the project design elevations. The top of levee elevations should be evaluated during inspections. Top of levee elevations should be checked at least every ten years or sooner as deemed necessary. If the PDT is expecting continuous settlement of the levee (see Chapter 8), topographic surveys of the top of levee should be considered to occur on a more frequent basis. The spacing between top of levee elevations points should not exceed 100 feet.

12.14.2 In addition, the project datums must be periodically assessed per ER 1110-2-8160.

12.15 Drilling in Embankment. Special requirements for drilling in the embankments shall be included in the O&M Manual. These requirements can be found in ER 1110-1-1807.

12.16 Flood Performance and Special Considerations. The PDT must identify the expected normal performance of the project during flood events, including the expectation for "normal" design performance that requires "flood fighting" activities (an example is the activities performed during high water events for a truncated seepage berm by the USACE Vicksburg District). The PDT must identify the effort and methods required to complete surveillance of the levee during flood events. The PDT should give indications of various measures of performance, such as seepage flow rates, and the corresponding magnitude and extent of flood fighting activities (for example, flood fighting could be required if localized pin boils and small sand boils are distributed broadly along the entire length of the levee system, with larger boils likely between river mile x and y). If instrumentation is in place, the PDT will identify threshold levels with corresponding actions required by the local sponsor. Any critical issues, such as seepage concerns, should be identified to the local sponsor.

12.17 Flood Fighting Expectations.

12.17.1 This section is intended for flood fighting of the levee - not flood response that may be performed within the community behind the levee. Any new or existing levee project must develop a surveillance, monitoring, and flood fighting procedures for monitoring levee performance during high water events and taking necessary actions such as flood fighting.

12.17.2 In some regions, flood fighting is considered a normal expectation and is included as an expectation during the development of the levee design, especially for water levels approaching the design water level. However, even if a levee is designed for satisfactory performance during a design event, the surveillance and monitoring should begin at a lower water level. For example, truncated seepage berms with heave/uplift factors of safety of 1 or less at the berm toe are expected to produce seepage boils, requiring increased monitoring, sand bagging, and other flood fighting activities to prevent progression of backwards erosion failure of the berm and levee. The local sponsor shall increase surveillance of the levee when the flood water elevation is above the ground surface elevation at the landside toe of the levee and undertake appropriate flood fighting activities, as required.

12.17.3 For flood fighting activities to be eligible for consideration in the risk assessment (such as including an “intervention” node in a potential failure mode analysis event tree per Chapter 1), the normal level of flood fighting must be described in the system O&M manual. Evidence of flood fighting activities in excess of this level, particularly for flood loadings below design levels, shall be considered evidence of poor performance, indicating design performance expectations are not being met, increasing the likelihood of future poor performance and potential breach, and likely warranting reevaluation of levee reliability.

12.18 Special Features Operations Manuals (closures, pump stations, culverts, etc.). Discussion and requirements for maintenance of special features should be included in the O&M Manual. See USACE guidance specific to the type of facility.

12.19 Routine Maintenance. Discussion and requirements for routine maintenance should be included in the O&M Manual.