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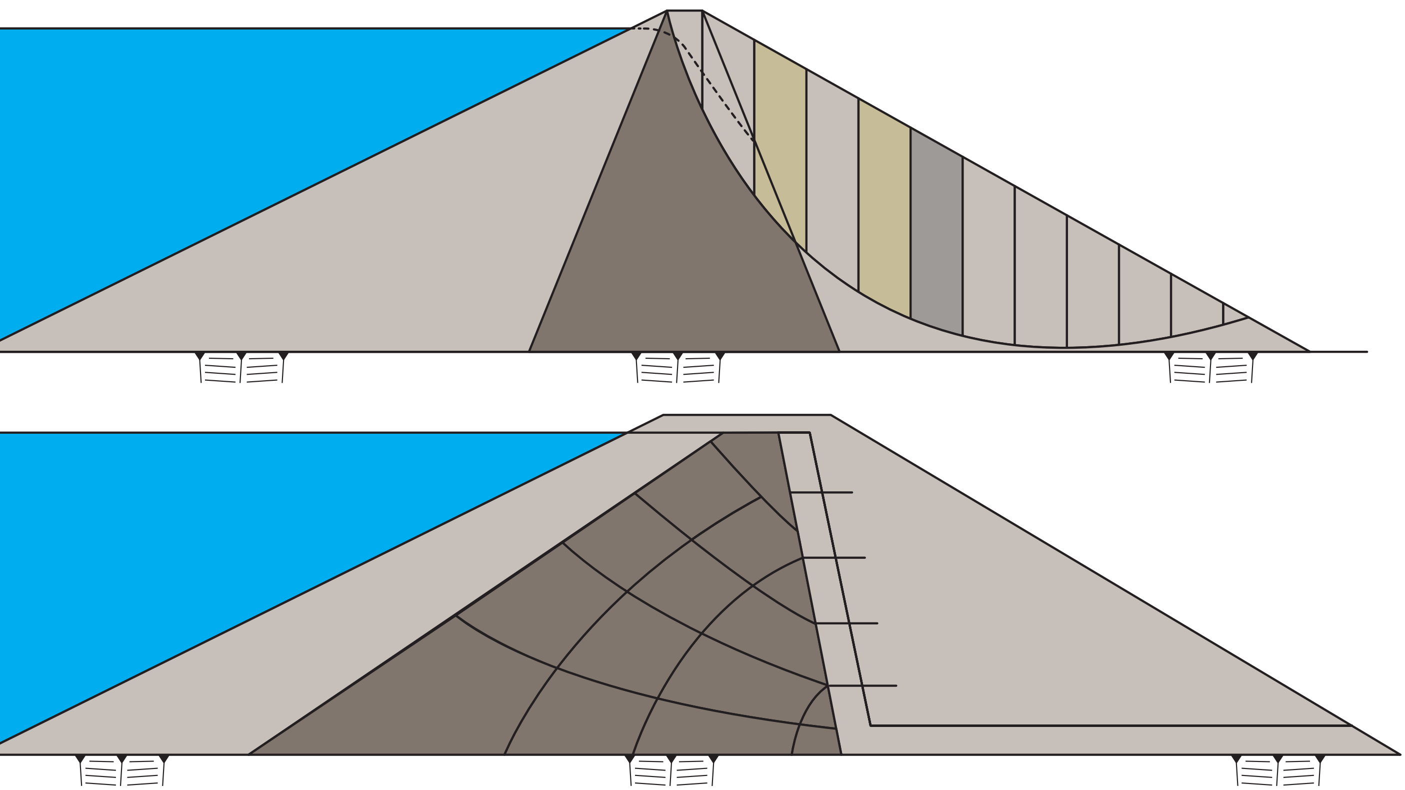
Seepage and Slope Stability Modeling for Embankment Dams

A Guidance Document for Planning, Interpreting, Verifying, and Reporting Results



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Prepared for:

State of Montana, Department of Natural Resources and Conservation
1424 9th Avenue
Helena, MT 59620

Prepared by:

AECOM
7595 Technology Way
Denver, CO 80237
aecom.com

Acknowledgements

This document is intended to provide practical guidance for dam owners and engineers on seepage and slope stability modeling of embankment dams, particularly small embankment dams. This document is not intended to be an all-inclusive guide for completing seepage and slope stability analyses for embankment dams. In many instances, the document directs readers to other references that provide more detailed information. In addition, an extensive list of references on the topic is provided at the end of this document.

This document was prepared by AECOM, under contract to the State of Montana, Department of Natural Resources and Conservation (DNRC). The work was authored by Julie Heitland, P.E. and Harry Donaghy, P.E., and was reviewed by Ms. Jennifer Williams, P.E. of AECOM and Ms. Michele Lemieux, P.E. of Montana DNRC.

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Acronyms and Abbreviations

1D	One-dimensional
2D	Two-dimensional
3D	Three-dimensional
ASDSO	Association of State Dam Safety Officials
CD	Consolidated-drained
cm/s	Centimeters per second
CPT	Cone penetrometer test
CU	Consolidated-undrained
DNRC	Montana Department of Natural Resources and Conservation
DS	Direct shear
DSS	Direct simple shear
EAP	Emergency Action Plan
FEMA	Federal Emergency Management Agency
FS	Factor of safety against sliding
ft^2/day	Square foot per day
ft^3/sec/ft	Cubic foot per second per foot
gpm/ft	Gallons per minute per foot
H:V	Horizontal to vertical
IDF	Inflow design flood
O&M	Operations and maintenance
OCR	Overconsolidation ratio
SHANSEP	Stress History and Normalized Soil Engineering Properties
SPT	Standard penetration test
S_u	Undrained shear strength
UC	Unconfined compression
UU	Unconsolidated-undrained

1. Introduction

Seepage and slope stability modeling are often proposed as part of an embankment dam evaluation. However, the modeling objective is often not well defined, and the model output may be nothing more than an expensive, colorful graphic without much insight into the dam's seepage- or stability-related issues, if not appropriately modeled, interpreted, and documented in an analysis report.

Nonetheless, seepage and slope stability modeling can help engineers, regulators, and owners better understand how seepage and stability may influence the performance of an embankment dam and provide the information needed to guide future dam safety actions. The following are some questions engineers, regulators, and owners should be asking before proceeding with a seepage or slope stability model:

- What should engineers consider when proposing a seepage or slope stability model to a dam owner or regulator?
- What should dam owners and regulators consider when reviewing an engineering proposal that involves a seepage or slope stability model?
- How do engineers develop an efficient model to achieve the desired objective?
- What are the minimum data needs for a reliable model? How much effort will the modeling take?
- How should the results be interpreted? How does one check for reasonableness of the results?
- What if two-dimensional models cannot adequately represent actual conditions?

The purpose of this guidance document is to provide a basic understanding of the standard of practice in preparing seepage and slope stability analyses of embankment dams. This document provides tips, tools, and guidance for planning the analyses, including modeling considerations, as well as interpreting, verifying, and reporting the results. Basic seepage and slope stability concepts are summarized throughout the document with references to additional publications that elaborate on the concepts. This document does not provide guidance on how to perform seepage and slope stability modeling but presents some basic modeling considerations.

The content of this guidance document is intended for:

- Entry-level to senior-level dam safety professionals and engineers,
- Dam owners,
- Dam regulators, and
- All other members in the dam safety community with interest in seepage and slope stability modeling of dams.

Information presented in this guidance document related to seepage analysis and modeling considerations was primarily adapted from two partnering technical papers prepared for the Dam Safety 2020 National Conference of the Association of State Dam Safety Officials (ASDSO) titled *Dam Seepage Models – Tools, Rules & Guidance (From a Regulatory*

Perspective) [Lemieux 2020] and *Seepage Models – Tips, Tools & Guidance (From an Engineer's Perspective)* [Heitland et al. 2020]. These papers were the steppingstones to this more comprehensive guidance document that has been prepared in conjunction with AECOM and the Montana Department of Natural Resources and Conservation (DNRC).

2. Seepage in Dams

2.1 What Is Seepage?

Seepage is the flow of water through the porous space within a soil or rock mass. In embankment dams, seepage can occur through the embankment, foundation, abutments, or along embankment penetrations. This includes flow through a large area of soil or concentrated flow along defects, such as cracks, loose lifts, rock discontinuities (e.g., fractures and joints), and other pathways. The reservoir is generally the largest source of water for seepage, but it may also come from groundwater sources. Figure 2-1 shows an example of seepage emanating from the downstream toe of an embankment dam.

Figure 2-1: Seepage Emanating from Downstream Toe of Embankment Dam (Photo Courtesy of the DNRC)

Seepage and leakage occur to some degree at all embankment dams and is not necessarily a problem if it is identified, monitored, evaluated, and controlled. Seepage can become a dam safety concern if it is not controlled and results in internal erosion, excess uplift pressures, or instability. Some factors that can lead to uncontrolled seepage include the following (FEMA 2015b):

- Hidden construction defects that originated during design and construction
- Large unprecedented seismic or climatic events
- Deterioration of one or more seepage control features
- Gradual deterioration of the embankment or foundation from past seepage
- Unabated animal burrow or tree growth activity

2.2 Seepage-Related Issues

This guidance document discusses the planning and interpretation of seepage analyses, which can provide insight into seepage-related issues. Several studies have been conducted on the failure of embankment dams (e.g., Richards and Reddy 2007; Foster et al. 2000). These studies consistently showed that seepage-related failures of dams comprise about one-half of all documented dam failures. Identifying potential seepage-related issues pertinent to a site prior to conducting a seepage analysis will help in establishing the objective, approach, and data needs of the analysis. The following is a list of some of the seepage-related issues that might be informed by seepage analysis:

- Internal erosion
- Uplift
- Increased pore pressures and slope instability
- Water loss (not a dam safety issue, but maintenance, nuisance, and economic issues)

Internal erosion occurs when seepage causes detachment and migration of soil particles. This soil movement can damage a dam's earthen embankment, foundation, or abutments. Internal erosion can develop in many forms and through many pathways. Figure 2-2 depicts an example of an internal erosion pathway.

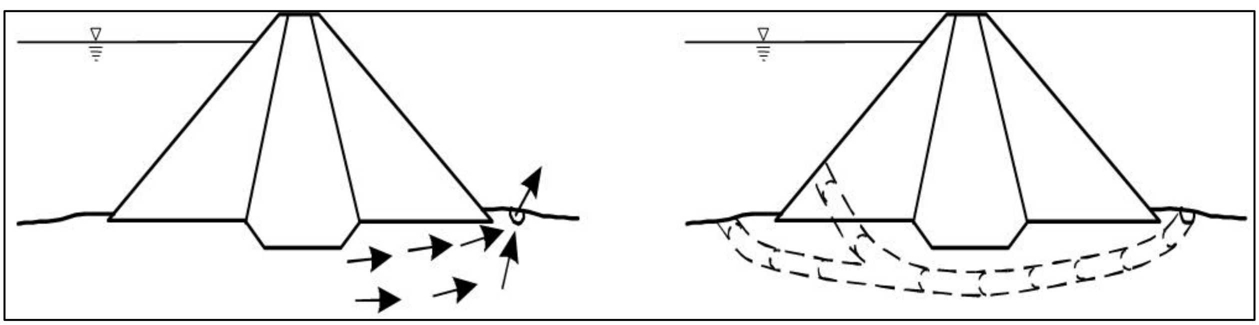


Figure 2-2: Example of an Internal Erosion Pathway through the Foundation of an Embankment Dam (Fell et al. 2008)

Several factors influence the susceptibility of a dam and its foundation to internal erosion, one of which is the magnitude of seepage forces that would be expected to occur under different loading conditions. Seepage analyses can be used to evaluate seepage gradients, velocity, and direction at various locations under anticipated loading conditions. For information on internal erosion pathways and mechanisms, refer to additional references in Section 7 (AECOM 2016; FEMA 2015a; FEMA 2016; Fell et al. 2008).

Figure 2-3), can cause damage to

concrete spillway chutes, and can cause instability in concrete structures or their foundations. Uplift pressures can be estimated by seepage analyses.

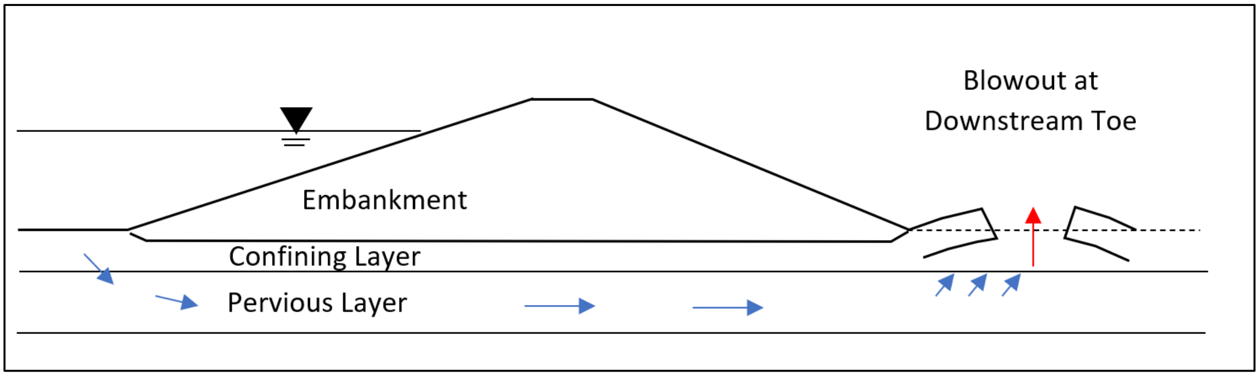


Figure 2-3: Example of Uplift Resulting in Blowout at the Downstream Toe of an Embankment

Figure 2-4 depicts slope instability in an embankment. Pore pressures in the embankment and foundation of a dam can be estimated from seepage analyses.

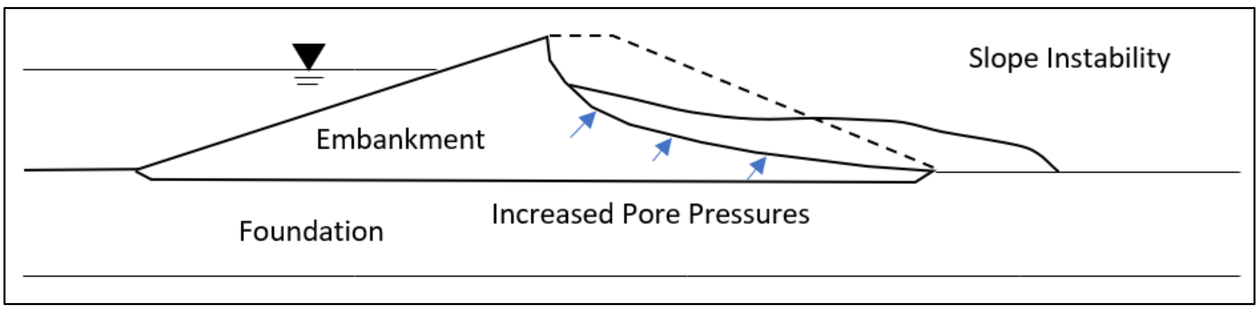


Figure 2-4: Example of Increased Pore Pressures in an Embankment Dam Resulting in Slope Instability

3. Seepage Analysis of Dams

A seepage analysis is a computational method that models the conditions of an embankment dam to estimate seepage characteristics through, beneath, and/or around the embankment. A seepage analysis can provide an understanding of the following:

- Seepage pathways,
- Seepage flow rates and velocities,
- Seepage gradients,
- Total head, pressure head, and pore water pressures, and
- Saturation.

There are a variety of seepage analysis methods for evaluating embankment dams that range from simple graphical approaches completed by hand to more complex numerical modeling using computer programs. Selecting an appropriate analysis method will depend on the objective of the seepage analysis and complexity of the situation. Common applications of a seepage analysis include quantifying the performance of the dam under current, expected future, and/or extreme conditions; evaluating observed seepage conditions; or evaluating and comparing seepage control design alternatives.

The results of a seepage analysis can be used for the following:

- Evaluating internal erosion potential,
- Evaluating slope stability implications, and
- Designing seepage control systems, including collection (filters and drains, toe drains, etc.) and reduction (low permeability blankets, cutoff walls, etc.).

3.1 Seepage Analysis Principles

The theoretical principles that govern the movement of energy through conducting media (such as electricity or heat) similarly apply to the movement of water through soils. Water moves from a higher energy state to a lower energy state. The differential in energy is the amount of energy required to overcome the soil's resistance to the flow of water. The theory used to understand and evaluate the seepage response of embankment dams is based on Darcy's Law and the Laplace equation. This section provides a basic overview of these principles for seepage analysis. For more information on seepage analysis principles beyond that discussed below, refer to additional references in Section 7 (FEMA 2015b; Reclamation 2014; USACE 1993; Cedergren 1989).

3.1.1 Darcy's Law

Similar to Ohm's Law, which governs the flow of electricity, Henry Darcy derived an empirical formula in 1856 to explain the behavior of seepage through saturated soils. Darcy's Law states that the amount of flow is directly proportional to the hydraulic gradient. Darcy's Law is expressed by the equation:

$$q = \frac{k\Delta hA}{L} = kiA = vA$$

Where: q = Rate of Seepage
k = Hydraulic Conductivity or Permeability
Δh = Head Loss
A = Cross-Sectional Area Normal to Direction of Flow
L = Length of Seepage Path
i = Hydraulic Gradient = Δh/L
v = Discharge Velocity = ki

Figure 3-1 is a schematic illustrating the concept of Darcy's Law, where a prismatic soil sample is exposed to a head of water on the left side and a smaller head on the right side. This results in water flowing through the soil sample at a rate directly proportional to the hydraulic gradient.

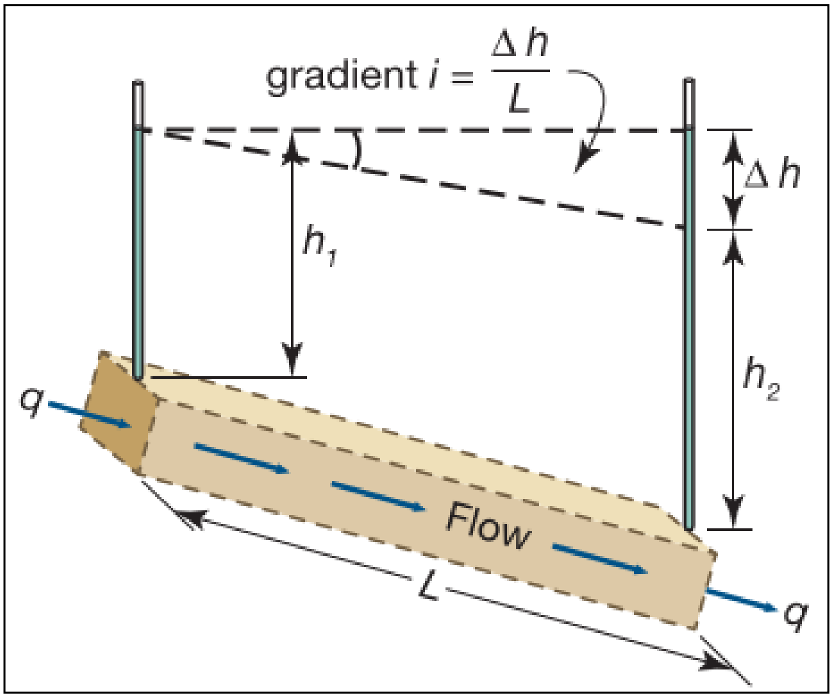


Figure 3-1: Schematic of Darcy's Law (FEMA 2015b)

Limitations to Darcy's Law include the following (FEMA 2015b):

- Darcy's Law is only applicable to laminar, steady-state flow through saturated, homogeneous soils.

- Darcy's Law is not applicable to flow through defects, such as cracks or rock fractures and joints.

3.1.2 Hydraulic Conductivity

As Darcy's Law states, the amount of flow is directly proportional to the hydraulic gradient. The constant relating the flow to the hydraulic gradient is the hydraulic conductivity (or permeability). Permeability is the ability of water to seep or flow through void spaces in soil. A high permeability material will pass more flow under the same gradient that a low permeability material will pass. Permeability is one of the most highly variable material properties in geotechnical engineering. Table 3-1 provides ranges of permeability values for a variety of soil types. This is a rough guideline which should only be used to compare permeability estimates as a reality check and not to be used directly in a seepage analysis.

Table 3-1: Typical Permeability Ranges by Soil Type (Cedergren 1989)

Soil Type	Permeability, k (cm/s)
Clays	$1x10^{-7}$ to $1x10^{-9}$
Very Fine Sands, Silts, Mixtures of Sand Silt and Clay	$1x10^{-7}$ to $1x10^{-3}$
Clean Sand, Clean Sand and Gravel Mixtures	$1x10^{-3}$ to 1
Clean Gravel	1 to $1x10^{2}$

Key factors affecting permeability include the following:

- Grain size and distribution,
- Soil structure,
- Density,
- Discontinuities/stratification, and
- Viscosity of fluid (not typically of consequence for embankment dams since the fluid is normally water).

Permeability is also heterogeneous (i.e., varies with location) and anisotropic (i.e., varies with direction of flow). For example, clay may have a low permeability, but fissures or desiccation cracks can provide a preferential flow path for water. In addition, alluvial soil deposits can lead to a higher permeability in the horizontal direction than the vertical direction. Figure 3-2 is a schematic illustrating heterogeneity and anisotropy.

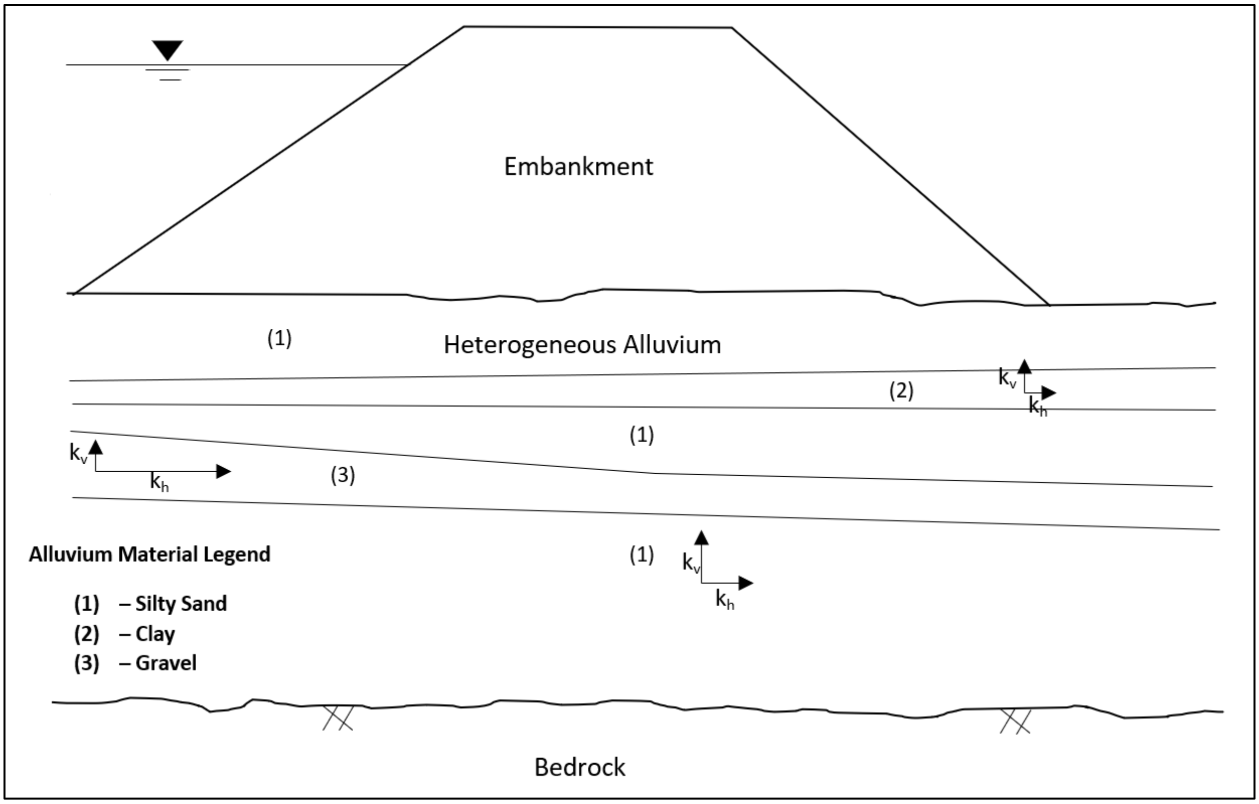


Figure 3-2: Schematic of Heterogeneous and Anisotropic Foundation

Figure 3-2 depicts a heterogenous alluvium foundation, sitting below an embankment dam, and overlaying bedrock. The alluvium foundation consists of a majority of silty sand, however there are seams of clay and gravel. This is a heterogenous foundation. Further, the gravel seam is anisotropic, meaning it has a higher hydraulic conductivity in the horizontal direction, than in the vertical direction.

Finally, permeability varies with the degree of saturation. As the degree of saturation decreases, the permeability can decrease by orders of magnitude.

Although there are several field and laboratory methods for estimating permeability, the effects of heterogeneity and anisotropy are difficult to identify and model with precision. Thus, selection of permeability values should be considered to give order-of-magnitude levels of accuracy.

3.1.3 Laplace Equation

The Laplace equation is a partial differential equation that describes the flow of water through homogeneous, isotropic soils. It is used in two- and three-dimensional (2D and 3D) seepage analyses. The equation assumes that the quantity of water entering an element must be equal to the amount leaving the element (i.e., steady-state flow). The Laplace equation can be arranged in terms of gradients and permeabilities by applying Darcy's Law. The Laplace equation for 2D and 3D flow is expressed by:

$$\text{2D: } k_x \frac{\delta^2 h}{\delta x^2} + k_z \frac{\delta^2 h}{\delta z^2} = 0$$

$$\text{3D: } k_x \frac{\delta^2 h}{\delta x^2} + k_y \frac{\delta^2 h}{\delta y^2} + k_z \frac{\delta^2 h}{\delta z^2} = 0$$

Where: k = Permeability in the x, y, and z Directions
h = Total Head

In two dimensions, the solution of the Laplace equation represents a family of curves that describe seepage flow. This family of curves is known as a flow net. Flow nets are presented in Section 3.2.1.1.

The Laplace equation for flow of water through soils requires the following assumptions (FEMA 2015b):

- The soils are homogeneous and isotropic (when soil is anisotropic, a transformation technique is used as discussed in USACE [1993]).
- The soils are saturated.
- The soil structure and water are incompressible (i.e., no change in void space).
- Flow is laminar.
- Darcy's Law is valid.

The Laplace equation is the mathematical basis used in seepage analyses. The Laplace equation in conjunction with specific boundary conditions and soil properties can be used to define seepage pathways, flow quantities, gradients, and pressures. Figure 3-3 is a schematic illustrating the concept of the Laplace equation.

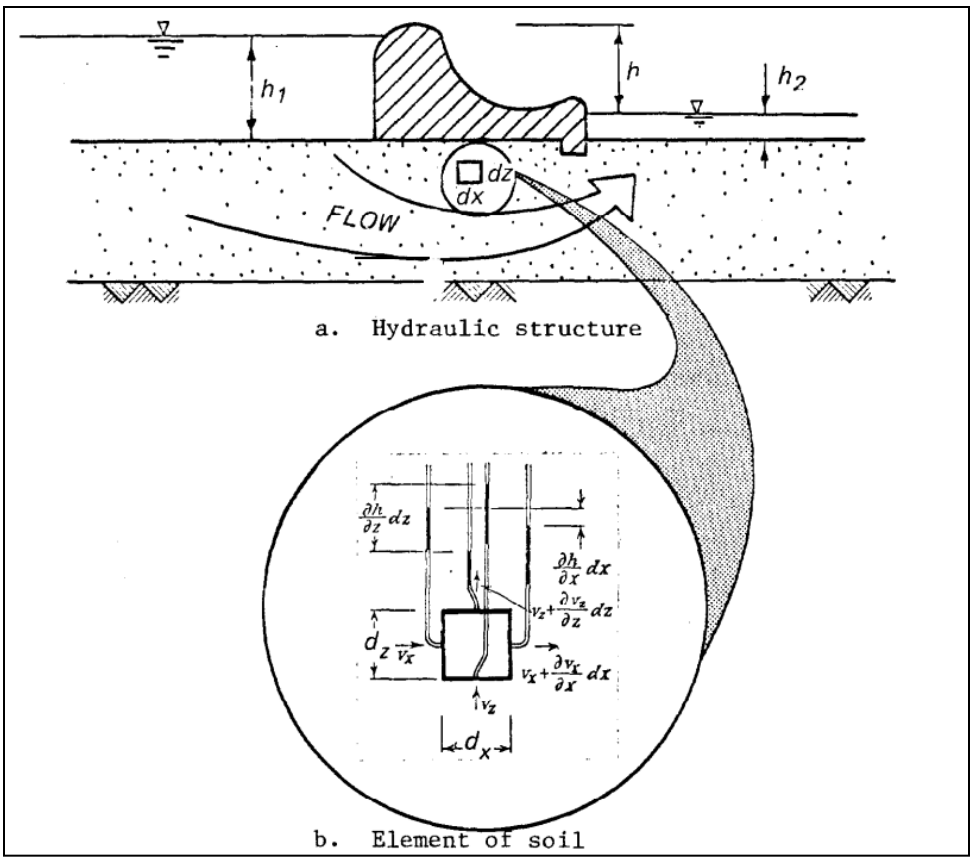


Figure 3-3: Schematic of (a) 2D Seepage Flow beneath Concrete Dam and (b) Steady-State Flow across an Element of Foundation Soil (USACE 1993)

3.2 Seepage Analysis Methods

As mentioned earlier, there are a variety of seepage analysis methods for evaluating embankment dams that range from simple graphical approaches completed by hand to more complex numerical modeling using computer programs. Most methods incorporate Darcy's Law and involve solving the Laplace equation. Typical seepage analysis methods include the following:

- Graphical Methods
  - Flow Nets
  - Graphical Construction of Phreatic Surface
- Analogs (Physical Models)
- Numerical Models

Generally, it is best to start with the simplest and least expensive method before proceeding to the more complex and costly method. However, modern and robust computing capabilities have made the use of numerical modeling the current standard of practice. This section provides a basic overview of the typical seepage analysis methods. General guidelines for selecting an appropriate seepage analysis method are provided at the end of this section.

3.2.1 Graphical Methods

Analyzing the seepage response of embankment dams using 2D graphical methods is the oldest approach and includes flow nets or graphically constructing the phreatic surface.

3.2.1.1 Flow Nets

Flow nets are a graphical solution of the Laplace equation that includes a family of curves that describe seepage flow. A flow net is constructed by hand and consists of two sets of orthogonal (i.e., intersecting at right angles) curves referred to as flow lines and equipotential lines. Flow lines represent seepage paths through the soil. The space between two flow lines is a flow channel, and each channel represents equal quantities of flow. Equipotential lines intersect the flow lines at right angles. Equipotential lines show the location of points within the soil that have the same piezometric head and represent equal pressure drops along the flow net. Figure 3-4 illustrates a flow net solution for seepage under a sheet pile wall in a permeable foundation that is assumed to consist of a homogeneous, isotropic soil. Figure 3-5 illustrates a flow net for unconfined seepage through a homogeneous embankment.

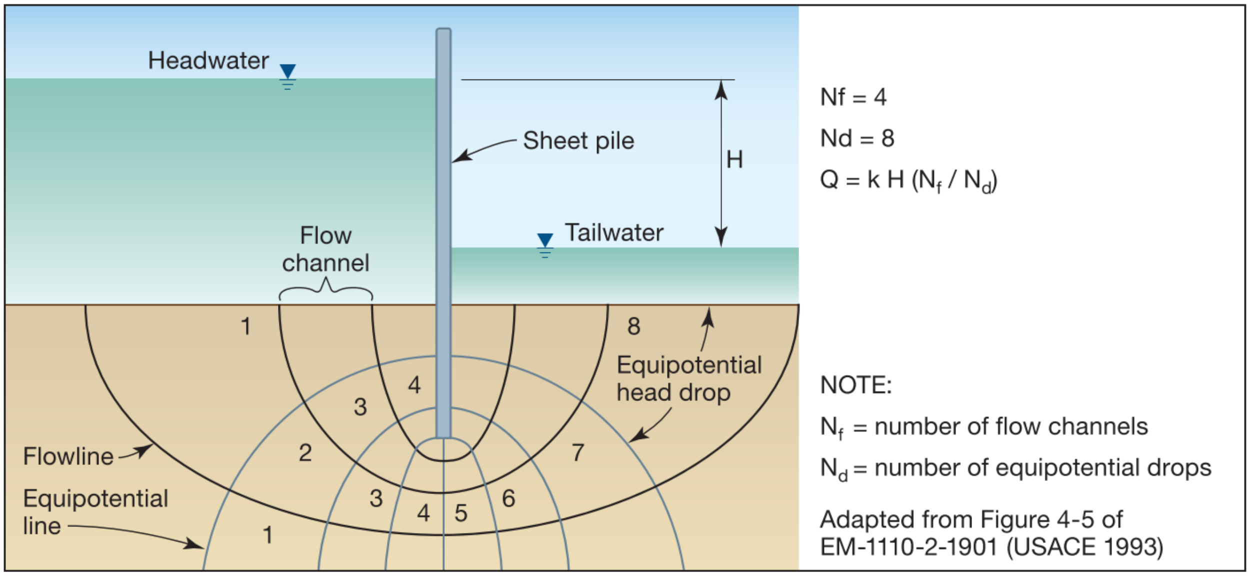


Figure 3-4: Flow Net for Seepage beneath Sheet Pile Wall in Permeable Foundation (FEMA 2015b)

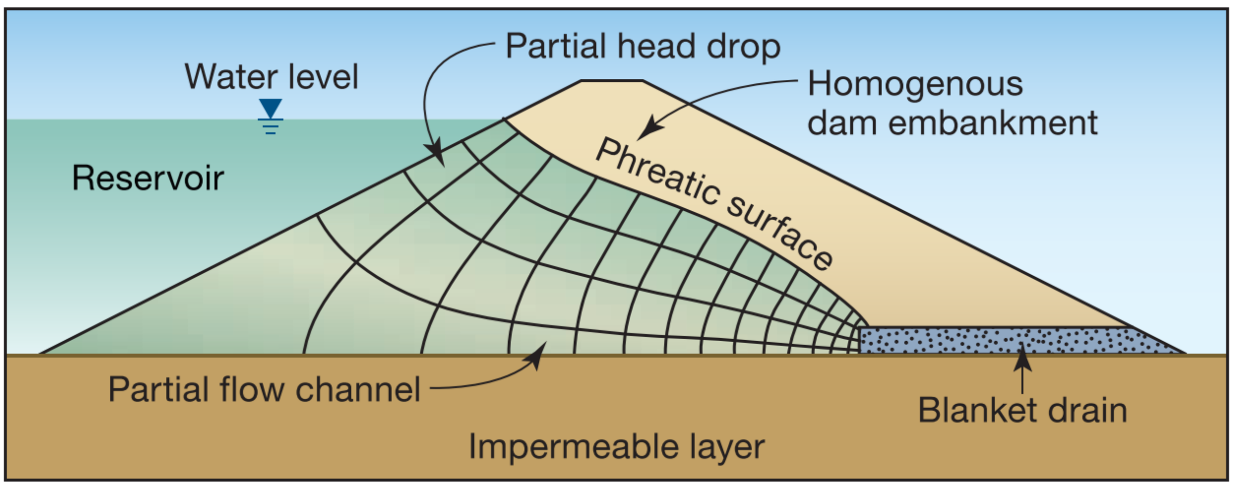


Figure 3-5: Flow Net for Unconfined Seepage through Homogeneous Embankment (FEMA 2015b)

Flow nets are a practical and versatile method for evaluating seepage and have historically been used to analyze 2D seepage problems. Flow nets are relatively fast to create, easy to draw for simple cases, inexpensive, and provide insight into seepage flow characteristics and quantities. However, flow nets take practice and experience to draw accurately, require a fair amount of simplification to geometry and material properties, are difficult to draw for complicated geometries and multiple permeabilities, and are no longer commonly used. With practice, flow nets can be a valuable tool for evaluating seepage in dams and can also be used to help verify numerical solutions. As discussed later, modelers should attempt to draw a conceptual flow net in advance of modeling, as it will help guide thinking and aid in model set up. For more information on flow nets, refer to additional references in Section 7 (Cedergren 1989; FEMA 2015b; NRCS 1973; NRCS 1979; Reclamation 2014; USACE 1993).

3.2.1.2 Graphical Construction of Phreatic Surface

The upper line of seepage (i.e., flow line) through an embankment dam is known as the phreatic surface and represents a line of zero pressure. The phreatic surface through an embankment can be graphically constructed using a procedure described by Casagrande (1937). Figure 3-6 illustrates the graphical construction of a phreatic surface for a homogeneous embankment.

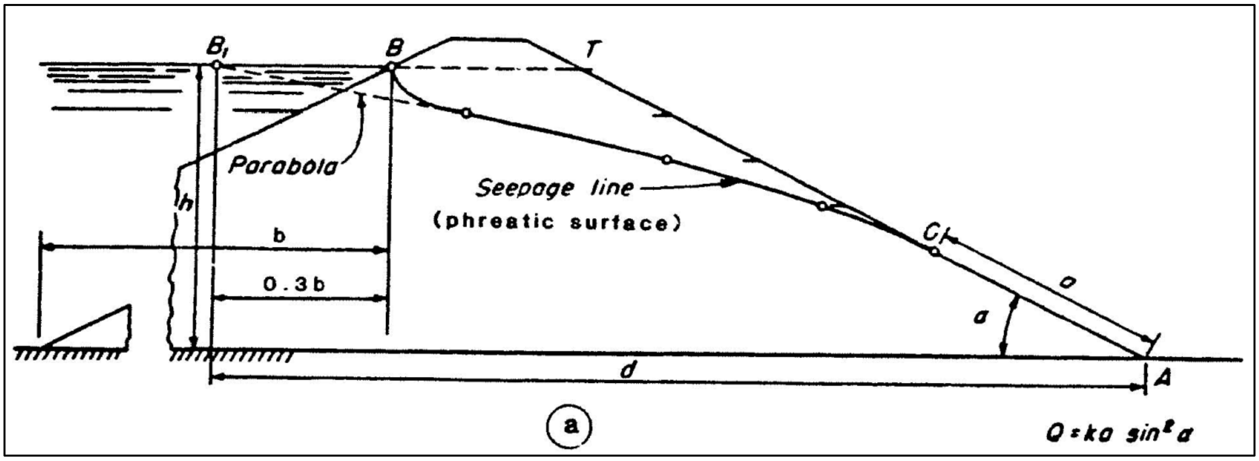


Figure 3-6: Graphically Constructed Phreatic Surface through Homogenous Embankment (Adapted from Reclamation 2014)

Graphically constructing the phreatic surface is a relatively fast, easy to follow, and inexpensive approach for defining the phreatic surface for slope stability analyses or estimating seepage quantities. This approach for constructing the phreatic surface is fast, repeatable, and may also be used as a starting point for construction of a flow net (NRCS 1979). However, it has limited applicability, less versatility than flow nets, and is no longer commonly used. Graphical construction of the phreatic surface is primarily limited to evaluating drainage alternatives for homogeneous embankments on relatively impervious foundations. For more information on graphically constructing the phreatic surface, refer to additional references in Section 7 (Casagrande 1937; FEMA 2015b; NRCS 1979; Reclamation 2014).

3.2.2 Analogs (Physical Models)

Before numerical models used computer programs, analogs were used to evaluate seepage flow. “Analogs” is a term for physical models that are analogous to the flow of water through porous media. Such physical models include electrical analog models and viscous fluid models. These models simulate the flow of seepage because the physics are governed by the same principles as flow. Figure 3-7 illustrates the use of an electric analog to evaluate seepage conditions beneath a concrete dam.

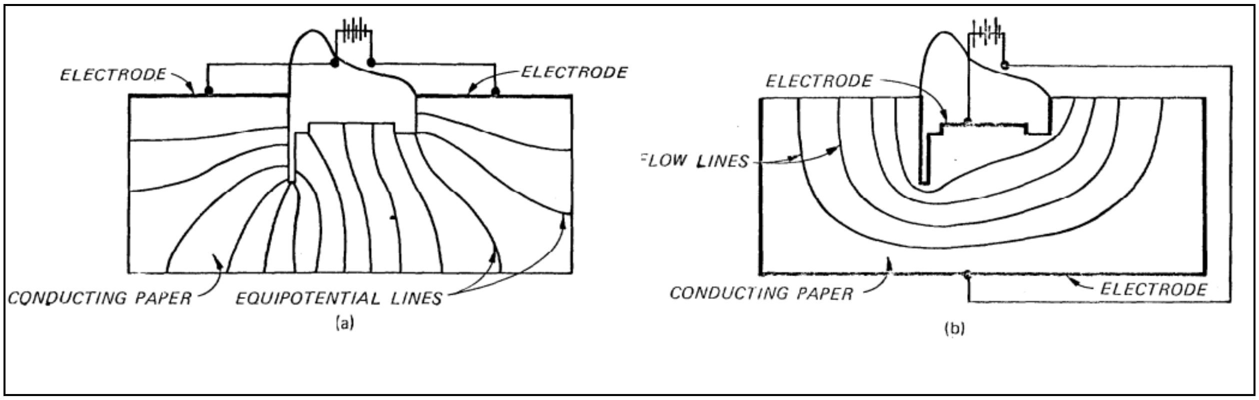


Figure 3-7: Use of 2D Conducting Paper to Evaluate (a) Equipotential Lines and (b) Flow Lines (USACE 1993)

Analogs can evaluate a variety of seepage problems and indicate the reaction of a system to a change in condition (e.g., change in head, geometry). However, analogs are rarely used and can be time-consuming and costly to calibrate depending on the type of model. For more information on analogs (physical models), refer to additional references in Section 7 (USACE 1993).

3.2.3 Numerical Models

The most common approach and current standard of practice for analyzing the seepage response of embankment dams is using 2D and 3D numerical models, provided that a robust analysis using modern computing capabilities is warranted.

3.2.3.1 General

Numerical models use computer programs to run finite element analyses that mathematically approximate the Laplace equation in complex flow conditions. In a numerical model, the geometry is discretized into small (i.e., finite) elements that form a grid. Each element intersection is called a node. The nodes represent a continuum through the entire model. The model uses a series of equations to approximate the Laplace equation. For example, if the grid consists of N elements, there will be N equations and N unknowns to solve. Figure 3-8 illustrates a numerical seepage model for a zoned embankment.

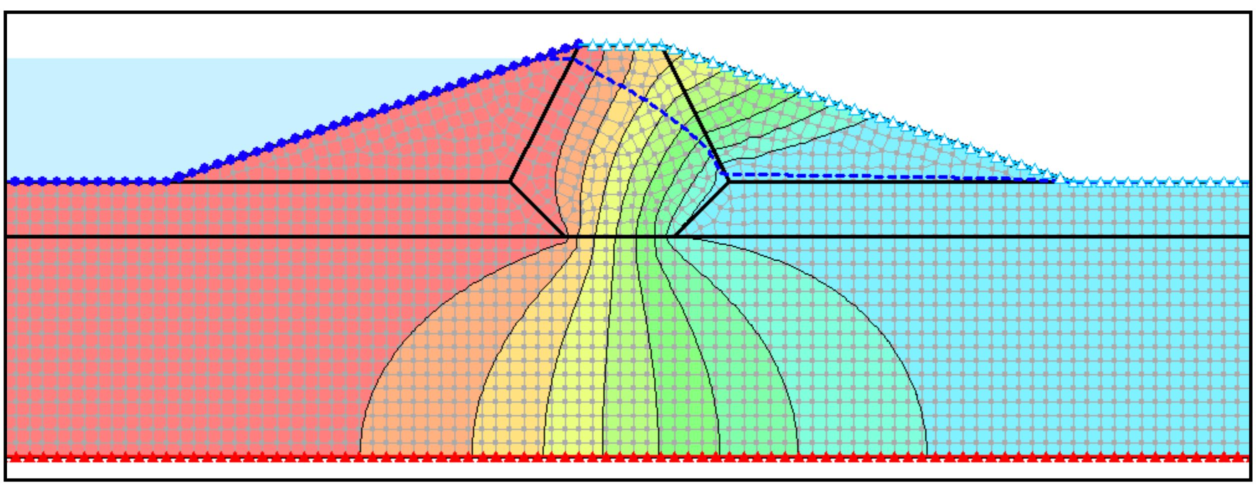


Figure 3-8: Numerical Seepage Model Results from the SEEP/W Program for Zoned Embankment (Heitland et al. 2020)

There are many benefits to numerical modeling, which include the following:

- Numerical models can properly characterize permeability and evaluate flow through both saturated and unsaturated soils. Characterizing unsaturated flow is a limitation of flow nets.
- Numerical models are easier to use for complex situations (e.g., complex embankment geometry or foundation stratigraphy).
- Both steady-state and transient (or time-dependent) flow can be modeled.
- Both 2D and 3D problems can be modeled.
- Zones where seepage gradients or velocities are high can be more accurately modeled by varying the size of the discrete elements.
- A variety of boundary conditions can be modeled.
- Most numerical models have graphical results that can be visually checked for reasonableness.
- Numerical models provide results (e.g., seepage flow rates, velocities, gradients, pressures) at any location (i.e., element) within the model.
- Results can be easily used and input into slope stability computer programs.

Some limitations to numerical modeling include the following:

- Numerical models are only as good as the modeler’s understanding of the input and ability to interpret the results.
- Modeling requires practice and training to understand the sensitivities of the model.
- Numerical models are susceptible to convergence issues.
- Numerical models will often run without error and produce professional-looking results that can be invalid or produce results that do not make sense. It takes knowledge of seepage principles and experience to properly interpret and verify the results.
- Modeling can be time-consuming and costly.

For more information on numerical models, refer to additional references in Section 7 (FEMA 2015b; GEO-SLOPE 2012; Heitland et al. 2020; Reclamation 2014; USACE 1993).

3.2.3.2 Computer Programs

Typical computer programs for modeling 2D and 3D seepage according to the Federal Emergency Management Agency (FEMA) are summarized in Table 3-2, along with their modeling capabilities, benefits, and limitations. The most commonly used computer program for seepage modeling in dams is SEEP/W developed by GEO-SLOPE, as part of their GeoStudio suite.

Table 3-2: Typical Computer Programs for Modeling Seepage (FEMA 2015b)

Computer Program	Method of Modeling	Modeling Capabilities	Benefits	Limitations
SEEP/W (GeoStudio)	2D and finite element 3D finite element capabilities were added in 2019	Groundwater, pore water pressures, seepage flow quantities, velocities, gradients, and uplift pressures.	• User friendly, good quality graphics. • Seepage pressures from SEEP/W can be imported into SLOPE/W for slope stability analysis.	• User friendly nature results in the program frequently being misused by novice analysts, which can result in unrealistic results. • Only models laminar flow through homogeneous media. Concentrated flow, such as through bedrock features, cannot be accurately modeled.
MODFLOW	2D and 3D finite element	Groundwater flow in aquifers; can evaluate well performance.	• Several user interfaces available from commercial and non-commercial sources.	• Non-orthogonal anisotropies not allowed.
FEFLOW	2D and 3D finite element	Groundwater and flow through porous and fractured media.	• Can model complex geologic features.	• More complex models are more expensive to run, require greater expertise, and take longer to learn how to use.
FRACMAN	3D finite element	Flow through fracture networks in bedrock.		• Requires detailed geologic information inputs.
FRAC	1D discrete boundary	Matrix and fracture flow through porous media and fractured rock.	• Works with MODFLOW to model flow through fractured rock.	• Requires detailed geologic information inputs.

In some situations, there may be merit for dam owners or regulators to develop a simplified seepage model to assist in making decisions. While seepage models can be valuable in understanding the performance of an embankment dam, the cost of purchasing a computer program can rarely be financially justified for owners and regulators. A new affordable option that is available is GEO-SLOPE's Basic SEEP/W, which is a trimmed down version of SEEP/W that is well suited for owner and regulatory needs. Limitations of the basic version include a coarse finite element grid and the inability to model transient flow. However, owners and regulators are not generally going to perform a rapid drawdown analysis or toe drain design. Rather, the owner or regulator is likely attempting to understand the embankment and foundation, general flow patterns, missing information, and most importantly, whether a more refined seepage model may be warranted. The complex seepage modeling is often best left to the consulting engineer.

3.2.4 General Guidelines for Selecting a Seepage Analysis Method

Deciding which seepage analysis method to use for the evaluation of an embankment dam is based on the objective of the analysis and complexity of the situation. Some methods will be appropriate for some situations, while others will not. General guidelines for selecting an appropriate seepage analysis method are summarized in Table 3-3.

Table 3-3: Guidelines for Selecting a Seepage Analysis Method (Adapted from FEMA 2015b)

Situation	Typical Investigations	Suggested Analysis Methods
Homogeneous embankment, impervious foundation, 2D steady-state	Phreatic surface, pore water pressures, seepage forces (stability), seepage control alternatives.	• Flow Net • Graphical Construction of Phreatic Surface • Numerical Model
Zoned embankment, impervious foundation, 2D steady-state	Phreatic surface, pore water pressures, seepage forces (stability), seepage control alternatives.	• Flow Net • Numerical Model
Homogeneous embankment, uniform pervious foundation, 2D steady-state	Phreatic surface, pore water pressures, seepage forces (stability).	• Flow Net • Numerical Model
	Seepage flow quantities, exit gradient, material property variations, seepage control alternatives.	• Numerical Model
Zoned embankment, pervious foundation, 2D steady-state	Same as above.	• Numerical Model
Uniform pervious abutment, 3D steady-state	Phreatic surface, seepage flow quantities.	• Plan View Flow Net[1] • Numerical Model
Heterogeneous pervious foundation and abutments, 3D steady-state	Phreatic surface, seepage flow quantities, exit gradient, material property variations, seepage control alternatives.	• Numerical Model
2D transient flow, steady boundary conditions	Tracking saturation, time to steady-state.	• Transient Flow Net[2] • Numerical Model
Non-steady 2D flow, saturated/unsaturated, zone or homogeneous foundation, transient boundary conditions, 2D transient state	First fill, flood cycle, cyclic operation, moisture content and pore pressure changes, effects of precipitation and evaporation.	• Numerical Model

[1] Plan view flow net described in Section 3.4.1.3.
[2] Transient flow net is described in Cedergren (1989).

3.3 When Is a Seepage Analysis Warranted?

A seepage analysis of an embankment dam can be used to gain a holistic understanding of the seepage regime through the embankment and its foundation. Seepage modeling can also provide valuable insight into how seepage may influence the performance of the embankment dam. However, seepage modeling can be time-consuming and costly and may not provide any additional insight into seepage-related issues. Key considerations in evaluating the cost-effectiveness and necessity for a numeric seepage model include the objective of the analysis, the amount of available information, and the complexity of the seepage regime. To identify when a seepage analysis may be warranted, consider the following questions:

- What is the objective of the seepage analysis?
- Can the objective be met adequately without a seepage analysis?
- Will a seepage analysis accurately capture the objective?
- Is there time to complete a seepage analysis (not an emergency)?

This section provides guidance in answering these questions to evaluate whether a seepage analysis may be warranted.

3.3.1 What Is the Objective of the Seepage Analysis?

This important question must be answered in detail before initiating a seepage analysis so that the analysis method and approach can be tailored to the specific objective. The objective of the seepage analysis will ultimately inform whether a model is warranted. The objective should be clear, well-defined, and consider the intended audience for which the objective will be explained.

When establishing the objective of a seepage analysis, it is important to have a basic understanding of instances when a numerical seepage model may or may not be warranted for an embankment dam. A few examples that may or may not warrant a seepage model are summarized in Table 3-4.

Table 3-4: Examples That May or May Not Warrant a Numerical Seepage Model (Heitland et al. 2020)

Examples That May Warrant a Seepage Model	Examples That May Not Warrant a Seepage Model
Evaluating an embankment under a future reservoir operating or flood loading condition it has not yet experienced.	Site with limited information on the embankment zoning, foundation conditions (including geology and stratigraphy), and material properties.
Evaluating an embankment with a complex geometry or foundation stratigraphy and/or a site with complex geology.	Well-instrumented site (e.g., piezometers) for which current data can be used to evaluate seepage response rather than a model.
Evaluating a non-hydrostatic condition (i.e., total head is not constant with depth).	A site that is not sensitive to seepage performance, which can be demonstrated using conservative assumptions.
Site configuration that limits accuracy of 2D simplified assumptions.	Sensitivity analyses (e.g., slope stability) can be performed for a potential range of pore water pressures to compensate for uncertainty.

When describing the need for a seepage model, the description must be written for both a technical and non-technical audience, limiting undefined jargon. Depending on the non-technical audience, it may be necessary to supplement the objective description with hand drawings or more detailed information. The importance of this cannot be understated. The objective must be clearly defined for a potentially wide audience, including inexperienced dam owners, regulators, elected officials, and board/commission members, as well as peers and colleagues.

As an example, consider the following objective: “A seepage model will help with the design of a toe drain.” This is a simply stated objective with no clear explanation of why or how the seepage model will help with the design of the toe drain. Instead, the objective should be elaborated as follows: “A seepage model is needed to understand the seepage pressures and gradients in the alluvial sand and gravel foundation layer and verify this layer is the likely primary water bearing layer. The model will be used to understand how these pressures and gradients change with distance from the reservoir and the potential for internal erosion of the foundation materials in this layer to initiate. It is suspected that the deeper bedrock is low permeability (tight) and not transmitting significant water; the model will help confirm this assumption. The model will also be used to confirm that a toe drain will be effective in intercepting flow through the water bearing layer and provide guidance as to where the drain should be located and its approximate depth. The model will be useful in evaluating performance of the proposed drain during a flood event that raises the reservoir level 3 feet.”

3.3.2 Can the Objective Be Met Adequately Without a Seepage Analysis?

In some cases, the specified objective can be met without a seepage analysis if the embankment dam has adequate geotechnical information (including boring logs) and is well instrumented with piezometers in the embankment and foundation. Key factors in making this determination include the following:

- Piezometers are properly located and isolated in zones or layers of interest. Piezometric measurements can often be used to evaluate the seepage response and phreatic surface in an embankment dam rather than a model.
- The reservoir level does not fluctuate significantly on an annual basis such that steady-state flow conditions can be assumed. It is more difficult to estimate seepage response from measured data if transient flow conditions exist.
- There is no need to evaluate the embankment dam under a future operating condition (e.g., higher reservoir pool level due to dam raise) or extreme loading condition (e.g., flood pool level). If there are questions about the embankment performance under future operating or extreme loading conditions, a model will be necessary.
- The dam owner has a limited budget. This is especially important if there is concern that a seepage model may not produce useful results, or if there is a known issue that may make calibration challenging. In general, dam owners prefer to spend their money on tangible items—something they can defend to their board or commission.

While the objective may not always warrant a seepage analysis, using a numerical seepage model to develop embankment cross sections and plot the piezometer locations and measured water levels can be valuable in understanding the embankment geometry, internal zoning, foundation contact and stratigraphy, and seepage regime, even if the model is never run. Hand drawing a rough flow net or drawing a water level contour map (in the case of a 3D model) is highly recommended, as it helps to conceptualize flow through the system and is important for efficient calibration. Developing a conceptual model representing the hydrological and hydrogeological conditions of the dam site before the actual modeling begins provides opportunity to evaluate the effort that would be required and expected outcomes from the model.

3.3.3 Will a Seepage Analysis Accurately Capture the Objective?

Seepage is sensitive to small localized variations of permeability, defects, anomalies, and fissures that are difficult to identify and model with precision, such that a high level of accuracy can be difficult to achieve in a seepage analysis. A seepage analysis should be considered to give an order of magnitude level of accuracy that is dependent on the estimated permeabilities for the embankment and foundation materials. Therefore, sufficient data on the embankment geometry, internal zoning, foundation contact and stratigraphy, and material properties is warranted for developing effective seepage models. In some cases, additional site exploration and investigation may be required to obtain the necessary data to perform a seepage analysis. Further information on the minimum data requirements for performing a seepage analysis is presented in Section 3.4.2.

A couple of specific conditions that are difficult to accurately capture in a seepage analysis include embankment defects (e.g., cracks or construction flaws) and geologic defects (e.g., rock discontinuities), which are often the root cause of seepage issues. For these conditions, direct observation and monitoring are the best methods of evaluation.

3.3.4 Is There Time to Complete a Seepage Analysis (Not an Emergency)?

A sudden, unexpected failure of an embankment dam due to seepage is unlikely to occur if the following conditions are true (FEMA 2015b):

- The embankment has been properly designed and constructed to the current standard of practice.
- The embankment has been maintained properly.
- Inspections are routinely performed by qualified personnel.
- An adequate amount of instrumentation is installed, monitored, and evaluated on a timely basis.
- Dam safety repairs are made as conditions dictate.

However, there are instances where these conditions may not hold true, and sudden changes in seepage conditions may require emergency action rather than an analysis of the seepage conditions. Signs of seepage conditions that may indicate imminent danger include the following:

- A whirlpool in the reservoir, particularly above the upstream embankment slope.
- Sudden or increased cloudy or muddy seepage.
- Frequent sand boils.
- Sudden sloughs on the downstream embankment slope.
- Sinkholes on the embankment.
- Cloudy discharge observed adjacent to an embedded structure (e.g., outlet conduit, spillway wall).
- Abrupt changes in piezometric water levels or seepage flow rates.

If any of these conditions are observed, a seepage analysis may not be warranted, and corrective action should be taken quickly under the guidance of an experienced dam safety engineer. The type of corrective action will vary for each of the above conditions and should be outlined in the dam's Emergency Action Plan (EAP). An EAP is a formal document that identifies potential emergency conditions at a dam and specifies actions to be followed by the dam owner to minimize loss of life and property damage. For more information on EAPs, refer to additional references in Section 7 (FEMA 2013).

If it is determined that a seepage analysis is warranted, then the following sections should be consulted for guidance on planning a seepage analysis, as well as interpreting, verifying, and reporting the results.

3.4 Planning for a Seepage Analysis

While seepage modeling can be beneficial in understanding the design and performance of embankment dams, it should also be understood that all models are wrong in some context. Models are merely an oversimplification or interpretation of reality and should be treated as such. A seepage model is only as good as the modeler's understanding of the inputs, how the inputs are used in the model, and the modeler's ability to interpret the results. Thus, modelers must be fluent in the model inputs, outputs, and seepage theory and recognize the model sensitivities to get the most out of a seepage model.

This section provides tips, tools, and guidance on planning for a seepage analysis and focuses specifically on numerical modeling using computer programs, which is the most commonly used method for evaluating the seepage response of embankment dams. Other seepage analysis methods are introduced in Section 3.2, with references to additional publications for further information on these methods.

Planning for a numerical seepage analysis involves defining the modeling approach and minimum data requirements. The planning process should also include considerations for model setup, calibration, and convergence. Guidance is provided below for each of these topics. Guidance for the interpretation, verification, and reporting of results is provided in Section 3.5.

3.4.1 Modeling Approach

The first step in performing a numerical seepage analysis involves defining the modeling approach, which includes considerations for the following:

- Geometry,
- Steady-state versus transient seepage analysis,
- 2D plane strain versus 3D modeling, and
- Saturated only versus saturated/unsaturated material model.

3.4.1.1 Geometry

The geometry of both the dam site and embankment should be taken into consideration in seepage modeling. In a 2D analysis, cross sections should be selected at locations where critical seepage conditions are expected and seepage results are required. This will typically include the maximum embankment section at a minimum. In a 3D analysis, an entire dam and surrounding area can be included in the model.

3.4.1.2 Steady-State versus Transient Analysis

Seepage through an embankment dam can be analyzed under steady-state or transient flow conditions. A steady-state seepage analysis represents the long-term operating condition of a dam. In a steady-state model, internal pore water pressures and flow conditions are computed for a given set of boundary conditions and are assumed to be steady (i.e., unchanging). This condition is typically evaluated with the reservoir at the normal operating pool level and is the most commonly analyzed condition.

A transient seepage analysis is one that is always changing. In a transient model, both initial and future boundary conditions must be specified to evaluate how long it takes for the embankment materials to respond to the given set of boundary conditions. Typical transient analysis scenarios include evaluating the wetting front rate through an embankment during the first reservoir fill after construction, the maximum reservoir drawdown rate to meet stability or other requirements of the applicable regulatory agency, the annual pore water pressure regime through an embankment that experiences yearly reservoir fluctuations, and/or the effect of flood loading (i.e., how long it will take to saturate the embankment and reach a steady-state condition). An example transient analysis is presented in Figure 3-9, which illustrates a rapid drawdown scenario though a zoned embankment dam. A transient seepage analysis may be completed in conjunction with a slope stability analysis to evaluate safe drawdown rates.

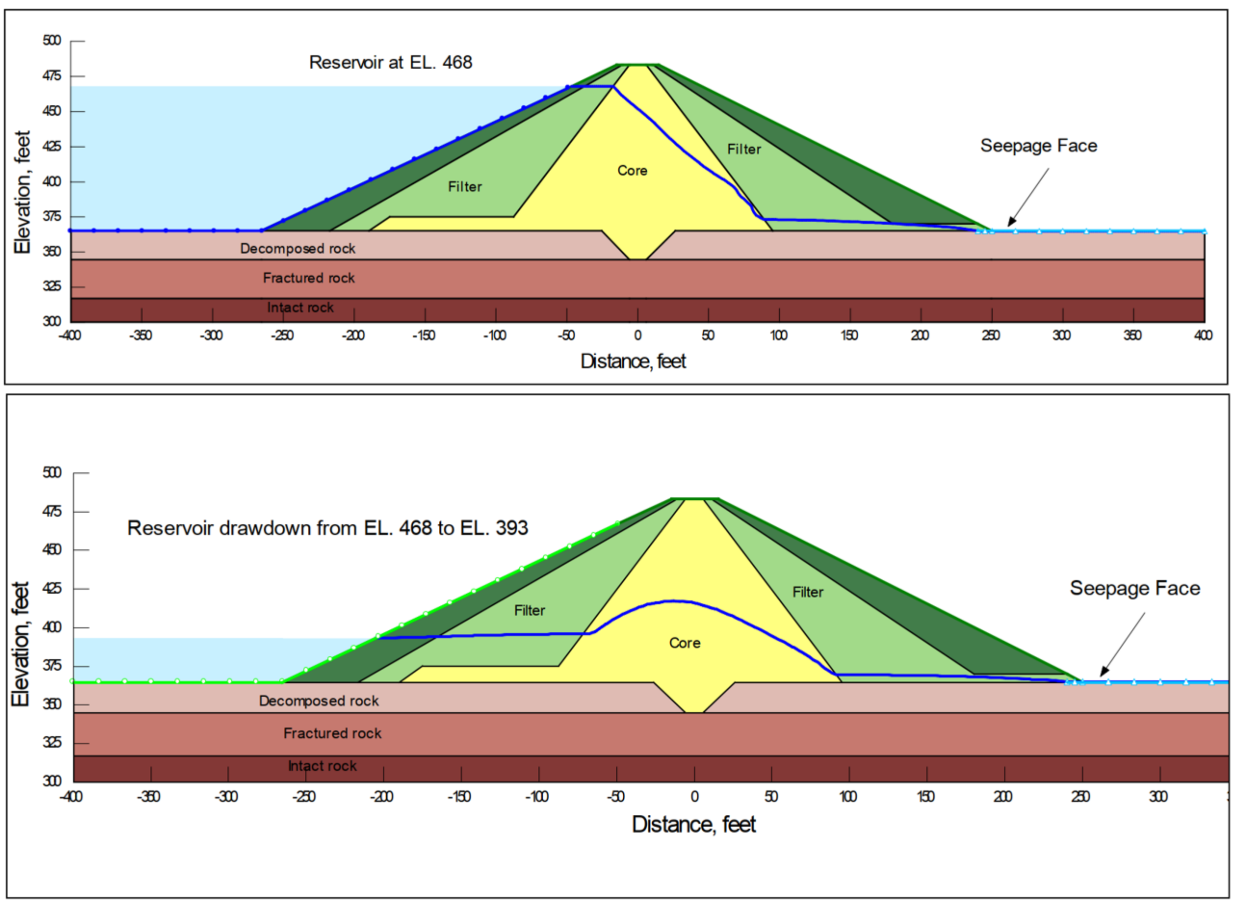


Figure 3-9: Transient Seepage Analysis for Drawdown through Zoned Embankment (Reclamation 2014)

Irrigation reservoirs that experience fluctuations in water levels annually may not be well represented with a steady-state seepage model. Typically, there is a lag between changes in reservoir water level and corresponding responses in piezometers. When modeling irrigation reservoirs, look carefully at lags in piezometric responses as the reservoir fills, reaches full pool, and starts to draw down. If a significant lag is obvious, it may be necessary to perform a transient seepage analysis to obtain meaningful results.

For more information on steady-state and transient seepage analyses, refer to additional references in Section 7 (Reclamation 2014).

3.4.1.3 Cross Section versus Plan View Models

Seepage analysis for embankment dams is most commonly modeled using 2D cross sections. However, in some cases a seepage analysis may be modeled in plan view. A plan view seepage model evaluates 2D groundwater flow using the same principles described in Section 3.2.1.1, Flow Nets. Although not commonly used, plan view seepage models have been used to evaluate groundwater flow around abutments or cutoff walls. More commonly, plan view seepage analyses are used in the selection and design of dewatering systems or relief wells.

A plan view analysis is intended to model groundwater flow through confined aquifers, so application to unconfined problems must be conducted with caution. For more information on plan view models, refer to additional references in Section 7 (GEO-SLOPE 2012; NRCS 1979).

3.4.1.4 2D versus 3D Modeling

Historically, seepage analyses have primarily comprised 2D modeling because of the complexity of 3D modeling and limited computer programs capable of 3D modeling. However, seepage analyses have recently been expanding into the 3D realm as the benefits of 3D modeling are becoming clearer and computer programs with 3D capability are becoming more prevalent.

While 3D seepage modeling can be valuable under certain conditions, 3D modeling is much more rigorous than 2D and therefore requires careful consideration to ensure 3D modeling is the appropriate approach. 3D modeling is very time consuming, requires a much greater level of effort and expertise, and is more costly. Conditions in which 3D seepage modeling becomes beneficial include the following:

- There are significant 3D cross-valley effects along the embankment dam alignment (e.g., narrow “v”-shaped valley profile, irregular/uneven or sloping foundation or abutment surface, pervious foundation or abutment) – Figure 3-10(a).
- A relatively long dam (or levee) that has a convex bend in the embankment. The bend serves as a point where seepage may converge from multiple directions – Figure 3-10(b).
- Complex embankment geometry (e.g., discrete seepage control features such as filters/drains, toe drain, relief wells) and/or foundation geology (e.g., bedrock discontinuities, faults) – Figure 3-10(c).

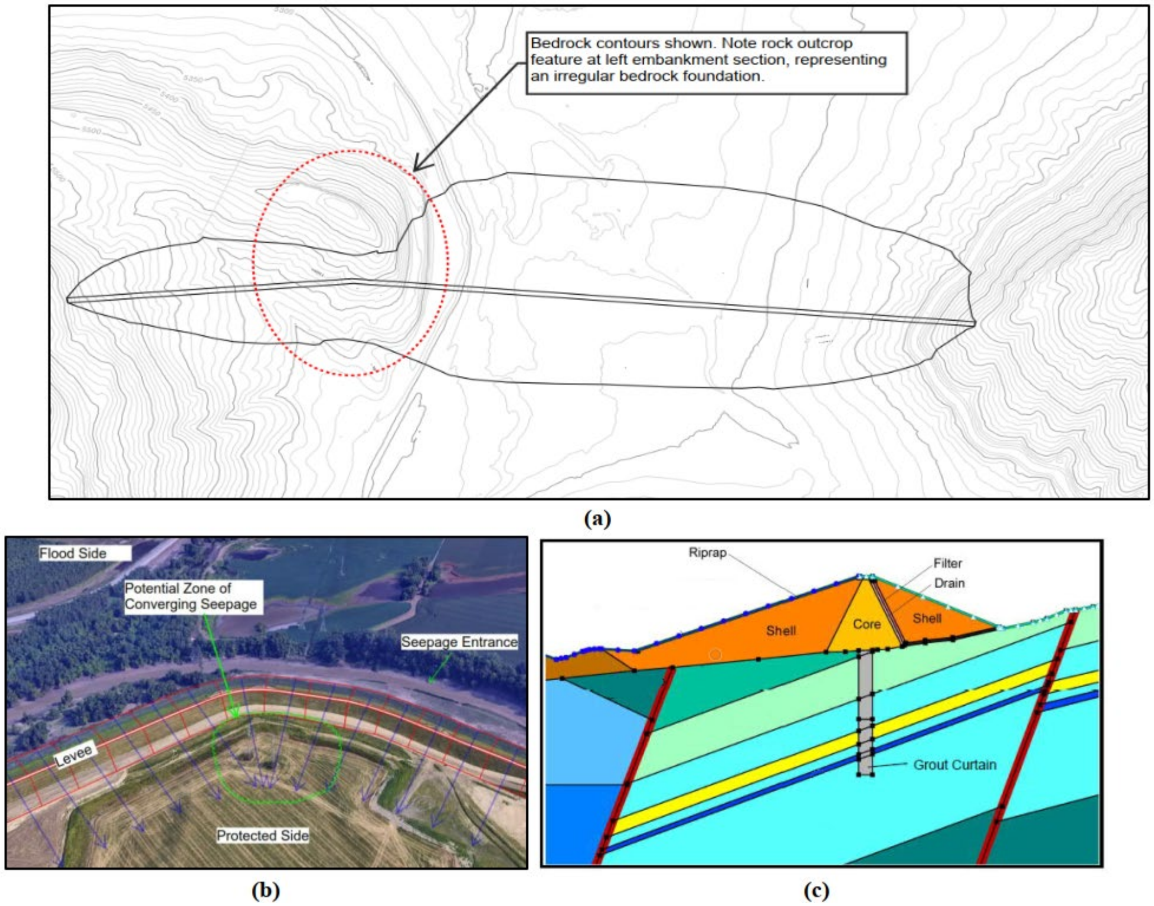


Figure 3-10: Examples of When 3D Seepage Modeling Becomes Beneficial: (a) Irregular Bedrock Foundation; (b) Convex Bend in Dam Alignment; (c) Complex Model Geometry (Heitland et al. 2020)

In all cases, there must be adequate information available to justify the expense of a 3D model. If lacking boundary condition input or geotechnical/geological data, a 3D model may not be warranted. It is recommended that any 3D model be informed by 2D models.

3.4.1.5 Saturated Only versus Saturated/Unsaturated Material Model

After the seepage analysis type (i.e., steady-state or transient and 2D or 3D) is established, the type of material model (i.e., saturated only or saturated/unsaturated) will need to be specified. A saturated/unsaturated material model should be selected to properly characterize permeability and evaluate flow through both saturated and unsaturated soils. This material model requires specifying hydraulic conductivity (permeability) and volumetric water content functions.

The hydraulic conductivity function describes the ability of the soil to transport water under both saturated and unsaturated conditions. The volumetric water content function describes the portion or volume of the pore spaces within a soil that remains water-filled as the soil drains. In a saturated soil, all the pore spaces between the soil particles are filled with water. In an unsaturated soil, the pore spaces are filled with air, becoming non-conductive conduits to flow. When the pore spaces become air-filled as the soil drains, the pore water pressures decrease rapidly and become increasingly more negative, which in turn quickly reduces the permeability of the soil. Thus, permeability becomes a function of negative pore water pressure when a soil is unsaturated, and this is captured by defining the hydraulic conductivity and volumetric water content functions. **Error! Reference source not found.** illustrate the relationship between the soil water content and hydraulic conductivity. For details on how to define and build the hydraulic conductivity and volumetric water content functions, refer to additional references in Section 7 (GEO-SLOPE 2012; Fredlund 1998; Reclamation 2014).

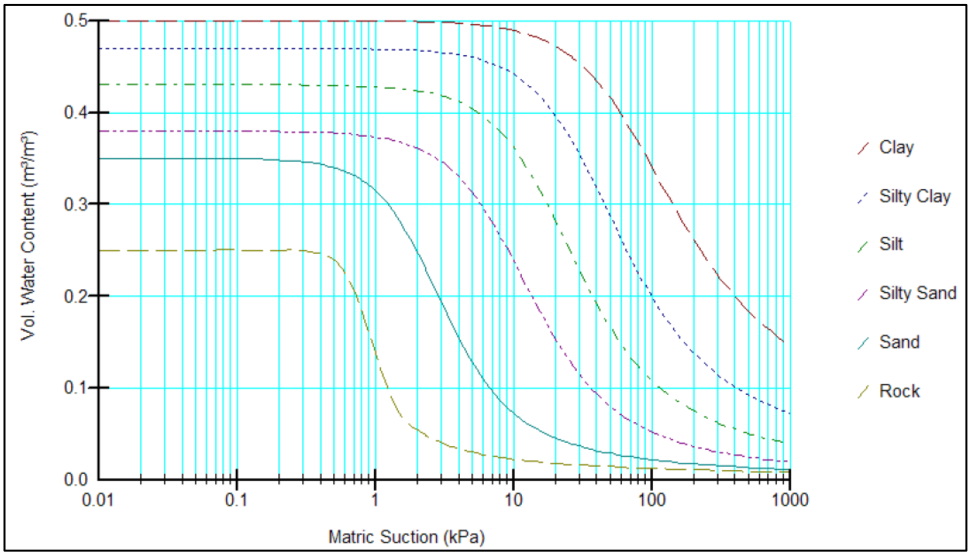


Figure 3-11: Typical Volumetric Water Content Functions of Soils (Reclamation 2014)

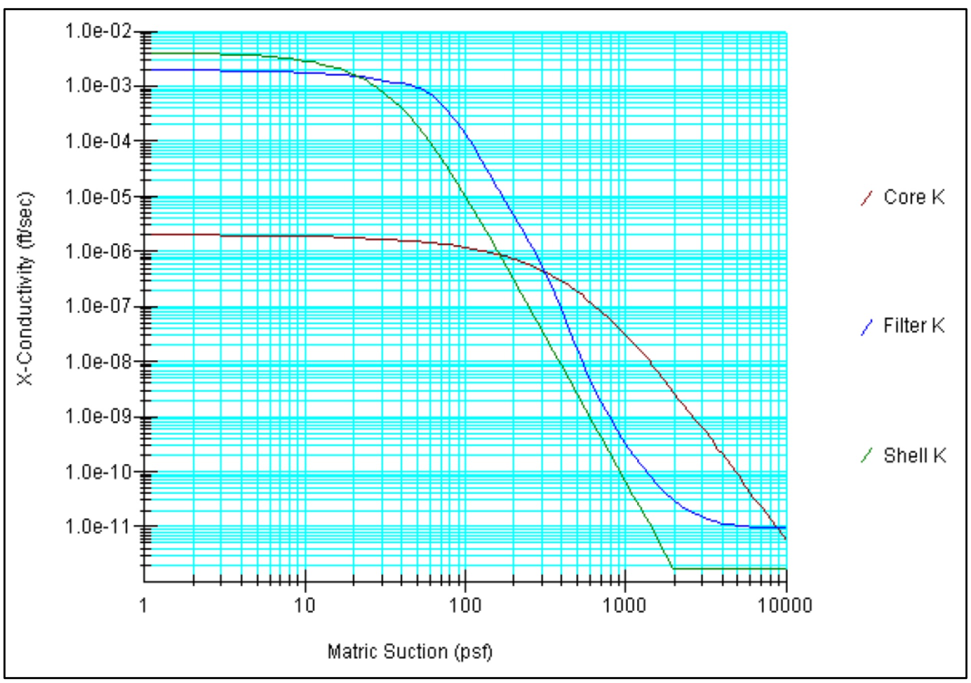


Figure 3-12: Example Saturated / Unsaturated Hydraulic Conductivity Functions of Embankment Soil (Reclamation 2014)

In most cases, a saturated/unsaturated material model should be specified because embankments typically have both saturated and unsaturated zones below and above the anticipated phreatic surface. This material model should also be selected if there is any uncertainty in whether specific materials comprising the embankment and/or foundation will remain unsaturated. A saturated-only material model should only be used when it is known that the materials will remain saturated under the given set of boundary conditions. For a coarse-mesh, simplified model, a saturated-only analysis could be used as an initial estimate of pore water pressures. However, a saturated/unsaturated analysis is recommended for almost all engineering design analyses.

3.4.2 Minimum Data Requirements

Seepage models can be adjusted based on very little data until the results look like what the modeler hoped they would, but "garbage-in, garbage-out" makes those models a waste of time. The minimum data needed to develop an efficient and reliable seepage model for an embankment dam are summarized in Table 3-5, along with the purpose of the data and types of reference documents where the data can be obtained. These minimum data requirements are discussed further below.

Table 3-5: Minimum Data Requirements for Seepage Modeling

Data Category	Data Requirements	Typical Data Sources
Model Geometry	• Embankment Geometry and Internal Zoning. • Foundation Contact and Stratigraphy.	• Topographic or Lidar Surveys • As-Built Construction Drawings • Design Drawings • Design or Construction Reports • Geologic and Geotechnical Investigation Reports
Material Properties	• Permeabilities for Embankment and Foundation Materials.	• Geotechnical Investigation and Data Reports • Construction Reports • Published Data
Calibration Data (for Existing Dams)	• Reservoir Levels, Piezometer Data, and Weir Flow Data.	• Instrumentation Records • Inspection Reports • Geotechnical Investigation and Data Reports

3.4.2.1 Model Geometry

Seepage models are typically developed for one or more embankment cross sections along the dam alignment. To develop the model geometry, there must be sufficient available data on the embankment geometry and internal zoning, as well as the foundation contact and stratigraphy. A detailed model geometry will delineate the various embankment zones (core, shells, filters, drains, etc.), foundation layers, and any other seepage control systems (toe drains, low permeability blankets, cutoff walls, etc.).

The best data source for defining the embankment geometry and internal zoning of an existing embankment dam is typically as-built construction drawings. Ideally, the external geometry (i.e., embankment crest, upstream and downstream slopes, and downstream ground surface) should be defined by a recent topographic or lidar survey. When construction drawings and recent surveys are not available, design drawings and/or design or construction reports can be used for defining the model. However, the modeler should be aware that the as-built and/or current embankment condition may differ from the design condition. Information from geologic and geotechnical investigation reports can also be valuable in verifying the internal zoning and/or variations in the embankment materials. For a new embankment dam that has yet to be constructed, the embankment geometry and internal zoning is typically defined using design drawings, but the design can be adjusted based on the results of the slope stability analysis.

The best data sources for defining the foundation contact and stratigraphy are typically geologic and geotechnical investigation reports.

3.4.2.2 Material Properties

Seepage modeling requires assigning material properties to the embankment and foundation materials. These material properties include permeability and the resulting anisotropy ratio (i.e., the ratio of horizontal to vertical permeability). Similar to the model geometry, there must be sufficient available data on the embankment and foundation materials to estimate the material properties. Typically, seepage properties are estimated using data collected from geotechnical investigations and/or published data. Information on the materials may also be available in construction reports.

Permeability values can be estimated from laboratory tests (e.g., constant head, falling head, or flexible wall permeameter), field tests (e.g., borehole soil permeability tests or rock packer tests), published tables of values, and/or empirical equations, which most often relate permeability to material gradation and void ratio (Cedergren 1989; NRCS 2009; Reclamation 2014). Laboratory and field tests represent the most reliable estimates of permeability. Published tables of values and empirical equations should be used with caution in regard to the accuracy of the estimated permeability values and should consider the specific material for which the empirical correlation is applicable. Most empirical correlations are applicable to granular materials and become unrepresentative for materials with high fines contents. Seepage models using material properties based only on published tables of values or empirical correlations should consider a range of potential permeabilities assigned to the various materials by performing sensitivity analyses to represent uncertainty. At a minimum, estimating reasonable seepage material properties requires adequate geotechnical/geological data on the embankment and foundation materials, including:

- Soil or rock type,
- Gradations,
- Density,
- Stratification/discontinuities, and
- Compaction procedures.

For more information on seepage material properties and laboratory and field permeability tests, refer to additional references in Section 7 (Reclamation 2014; USACE 1993).

For saturated/unsaturated material models, hydraulic conductivity and volumetric water content functions need to be specified. Depending on the level of accuracy required in seepage modeling, specific functions can be developed for the materials using soil-water retention properties. These properties can be measured in a laboratory or in the field (e.g., advanced tensiometers). However, measuring soil-water retention properties in the laboratory or field can be expensive. Soil-water retention properties can be estimated from grain-size distributions and compared to a database of existing test results (Fredlund 1998). For a screening-level solution, typical functions available in computer programs (e.g., SEEP/W) or published data can be used. In this case, sensitivity analyses on the functions should be performed to understand the effect of their variation and represent the uncertainty due to a lack of specific test data.

3.4.2.3 Calibration Data

For existing dams, seepage models should be calibrated if possible. Developing a plan to calibrate the model is an important step in the data review process. Calibrating a seepage model requires adequate records on reservoir levels, piezometer data, and/or weir flow data. Reservoir levels in conjunction with piezometer data can be used to calibrate the phreatic surface or potentiometric surfaces modeled within an embankment. Weir flow data can be used to compare actual measured seepage flow rates with estimated seepage flow rates. Caution should be used when relying on weir flow data to calibrate the seepage model, as weirs can collect water from other groundwater sources, or seepage may not be collected by the weir. Calibration data including reservoir levels, piezometer data, and weir flow data can typically be found in instrumentation records and/or inspection reports. Geotechnical investigation reports can also be reviewed to evaluate if groundwater was encountered during test hole explorations. The water levels measured in test holes can also be used to supplement piezometer data for calibrating the modeled phreatic surface.

3.4.2.4 Desktop Review

A desktop review of all available information for an embankment dam should be completed to inform whether there is sufficient data on the embankment geometry, internal zoning, foundation contact and stratigraphy, and material properties to perform the seepage analysis. The desktop review can also identify potential data gaps. FEMA (2015b) presents the following checklist for guidance on conducting a desktop review:

Desktop Review Checklist					
Design analyses and assumptions (original and any modification or remedial designs)	☐Yes	☐No	Photographs	☐Yes	☐No
			As-built drawings	☐Yes	☐No
Original geologic and geotechnical reports	☐Yes	☐No	Operation and maintenance records	☐Yes	☐No
Any data related to seepage analysis, filter design, stability analyses	☐Yes	☐No	Instrumentation records (reservoir levels, piezometric data, and weir flow)	☐Yes	☐No
Construction plans and specifications	☐Yes	☐No	Past inspection reports	☐Yes	☐No
Construction reports, logs, and records	☐Yes	☐No	Any correspondence from regulators relative to deficiencies or problems	☐Yes	☐No

Other questions to consider when evaluating whether there is sufficient data to develop a seepage model include the following:

- Are the embankment geometry, internal zoning, and foundation contact and stratigraphy understood and characterized well enough to develop a representative model? There must be adequate geotechnical/geological data to understand the embankment and foundation materials, as well as their material properties and spatial variability.
- Are construction documents available that define compaction procedures, which can also influence material properties (specifically anisotropy ratio)?
- Are there quality reservoir level measurements? Verify that the dam tender is taking accurate measurements. There must be consistent and plentiful data from a variety of reservoir levels.

- If there are existing piezometers, are they properly isolated within zones or layers of interest? If a piezometer's influence zone (screened interval plus filter pack) spans more than one zone or layer, it may be of limited use for model calibration.
- Is there hydraulic information on the downstream conditions (i.e., downstream piezometer measurements or tailwater levels) to establish downstream boundary conditions?
- Are there unusual flow patterns from abutments or other issues that could be obstacles in calibrating a model in a reasonable time period?
- Will there be too much guessing? The type of information needed will depend on the objective of the seepage analysis. For example, if the objective is to understand the seepage regime through an embankment during spring filling and only sporadic reservoir level measurements are available, it may be best to conscientiously collect a full year of data before attempting a seepage model.

The following is an example of how a modeler may plan for an analysis by performing a data review and comparing the available data to the data required for meeting the seepage analysis objective:

> Problem – After a recent flood event, shallow sloughing was observed on the downstream slope of a homogeneous embankment. It was decided that a slope stability analysis of the embankment should be performed. The embankment includes a downstream toe drain.
>
> Objective – Complete a seepage analysis to evaluate the phreatic surface for use in a slope stability analysis. The stability analysis will inform the risk of dam failure by slope instability.
>
> Data Requirements – The seepage model will require an accurate 2D cross section to be developed, for which the surface topography, embankment zoning, and foundation boundary need to be understood. The location and details of the toe drain are also significant to the model development. In addition to model geometry considerations, the material properties (specifically permeability) of the embankment and toe drain materials are needed. The foundation is considered to be impervious. Finally, reservoir operations and instrumentation monitoring data will be used to develop boundary conditions and calibrate the model.
>
> Results of Desktop Review:
>
> - Present-condition and preconstruction topographic surveys of the site and construction drawings were identified and will be used to develop the model geometry (including defining the foundation boundary and toe drain).
> - An operations and maintenance (O&M) manual, instrumentation monitoring records, and inspection report were also identified that include reservoir operations (normal operating pool level, flood pool level, etc.), as well as seepage weir data and the approximate location of the slough. This information will be used to develop boundary conditions and calibrate the model. Boundary conditions and model calibration are discussed further in Sections 3.4.3.2 and 3.4.4, respectively.
>
> A geotechnical data report with boring logs and material properties was not identified in the desktop review, so a data gap exists for the seepage properties. Permeability values could be reasonably estimated using the seepage weir data and location of the slough to calibrate the model. However, a geotechnical

investigation may be warranted to obtain material properties for the slope stability analysis. Therefore, the modeler must decide if additional exploration is warranted prior to developing the seepage model.

For information on performing geotechnical investigations in embankments, including typical drilling and sampling methods, refer to additional references in Section 7 (AECOM 2014a; FEMA 2015b).

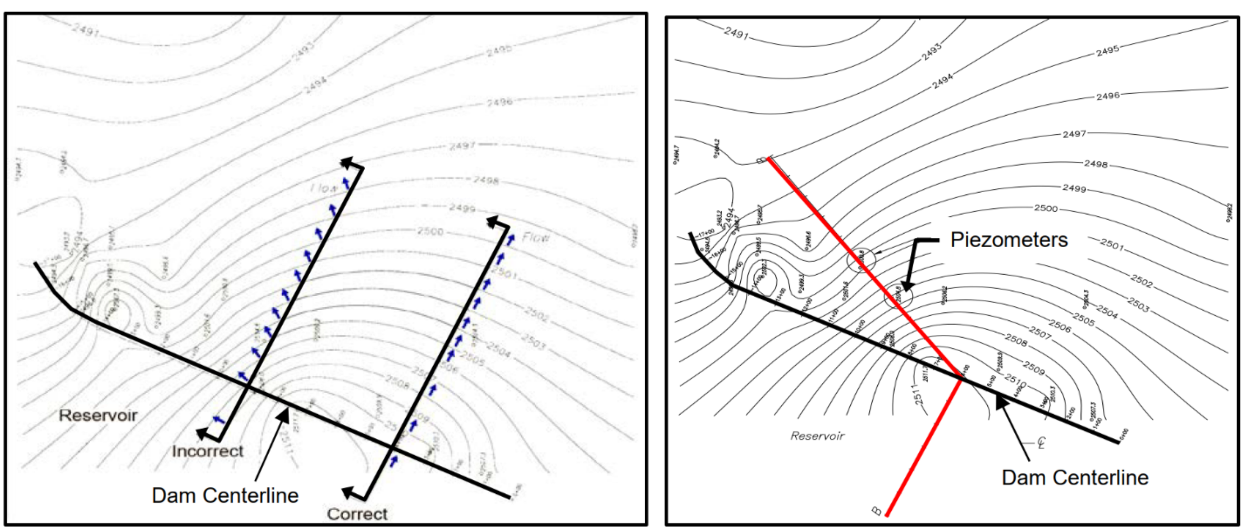
3.4.3 Model Setup Considerations

Three critical considerations for setting up a seepage model include cross section orientation, boundary conditions, and finite element mesh size and are discussed further below.

3.4.3.1 Cross Section Orientation

When performing a 2D seepage analysis, the cross section orientation must be selected carefully to limit 3D effects. A 2D cross section assumes all seepage flow is parallel to the section. Provided below are tips for consideration.

- Tip 1 – Orient the cross section parallel to the anticipated direction of seepage flow, as shown by the “correct” cross section in Figure 3-13(a). Figure 3-13 shows total head contours in plan view along a dam alignment. The “correct” cross section is oriented perpendicular to the total head contours, so that flow remains parallel to the section. The “incorrect” cross section intersects the total head contours at an angle such that seepage would be flowing out of the model in a third dimension. If flow is oblique to the section, results will not be a good match to reality.
- Tip 2 – Consider a bent section for cases where seepage flow may not be perpendicular to the dam alignment, as shown in Figure 3-13(b). The bent section remains perpendicular to the total head contours.
- Tip 3 – If calibrating the model using piezometer data, it would be prudent to select a cross section at or near piezometers.



(a) (b)

Figure 3-13: Cross Section Orientation for Numerical Seepage Modeling (Heitland et al. 2020)

3.4.3.2 Boundary Conditions

Boundary conditions must be carefully defined along the exterior boundaries of the seepage model, as the results are a direct response to the boundary conditions. Boundary conditions define where flow enters and exits the model, the upstream and downstream water levels, and even the direction of flow. Table 3-6 summarizes the typical boundary conditions assigned to a seepage model. These boundary conditions are also shown in Figure 3-14. Provided below are tips for consideration.

- Tip 1 – Input relatively long seepage models to avoid boundary effects at the vertical edges of the model, which can result in incorrect estimates of total head. The rule of thumb is to extend the model both upstream and downstream by a length equivalent to at least the width of the embankment section from the upstream to downstream toe.
- Tip 2 – Consider potential impacts of a no-flow boundary condition at the base of the model for an embankment founded on deep pervious materials. If the base of the model is too shallow with a no-flow boundary condition assigned, it can result in inaccurate estimates of total flow through the pervious foundation. For seepage models with deep, pervious foundations, the rule of thumb is to extend the base of the model below the base of the embankment to a depth between one and two times the height of the embankment.
- Tip 3 – In addition to the exterior model boundaries, boundary conditions (or nodes) can be applied to features within the model. For example, a zero-pressure node can be assigned to a discrete toe drain so that the phreatic surface intersects the toe drain.

Table 3-6: Typical Seepage Model Boundary Conditions (Heitland et al. 2020)

Model Boundary	Boundary Condition
Upstream Vertical Edge of Model Upstream Ground Surface Below Reservoir Level Upstream Embankment Slope	• Total head boundary equivalent to the elevation of the reservoir level
Embankment Crest Downstream Embankment Slope Downstream Ground Surface	• Potential seepage face boundary (i.e., where seepage can exit the model)
Downstream Vertical Edge of Model	• Total head boundary equivalent to either the elevation of the tailwater level (if applicable) or the elevation of the natural groundwater level • If there is no tailwater or no downstream piezometer to measure the groundwater level, the total head boundary is often assumed to be equivalent to a groundwater level at the downstream ground surface elevation. If the model results indicate this assumption is influencing the seepage patterns at the dam, then the extent of the model domain should be extended farther downstream.
Base of Model	• No-flow boundary

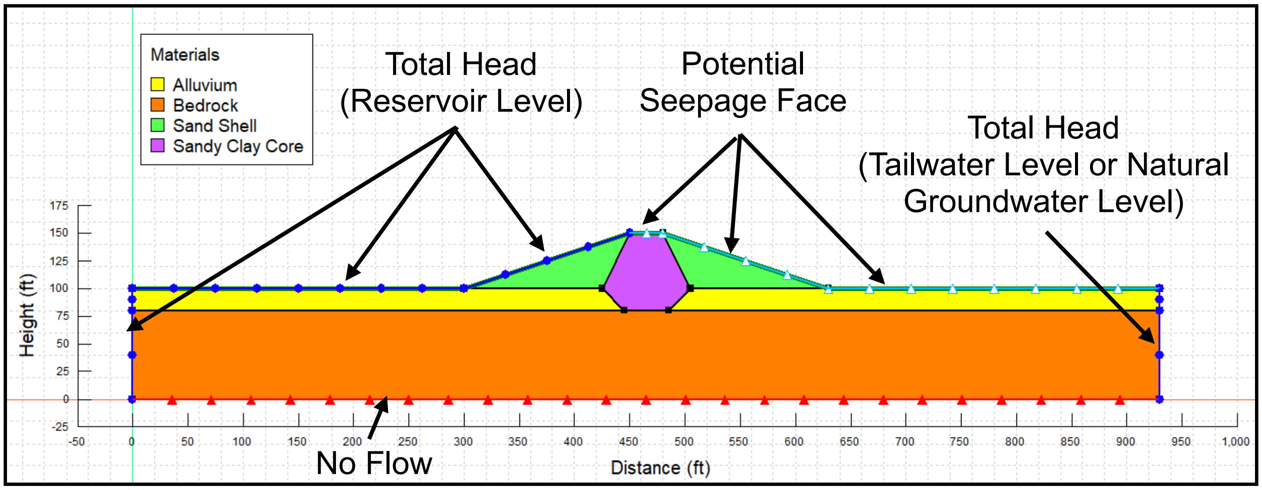


Figure 3-14: Typical Seepage Model Boundary Conditions from the SEEP/W Program (Heitland et al. 2020)

Information to support boundary conditions can typically be found in O&M manuals (e.g., annual reservoir fluctuations, normal operating pool level, flood pool level), instrumentation monitoring reports (e.g., piezometer and seepage weir data), or inspection reports.

3.4.3.3 Finite Element Mesh Size

The mesh size is dependent on the specific seepage model. Typically, a larger mesh size is specified for a large model geometry and a smaller mesh size is specified for a small model geometry. A common mistake is thinking that the more finite elements there are, the more accurate the results will be. Assigning a very small mesh size to a large model geometry can cause the model to run unreasonably slow, making it difficult to obtain results. Provided below are tips for consideration.

- Tip 1 – Start with a large mesh size (fewer elements) and verify that the results make sense. Once verified, reduce the mesh size as appropriate to better define the seepage regime. This will require several iterations to understand the model sensitivity to mesh size. Figure 3-15 and Figure 3-16 illustrate a comparison between two different mesh sizes, with Figure 3-15 showing model results using a 10-foot mesh size and Figure 3-16 showing results using a 5-foot mesh size. The larger mesh size results in an irregular phreatic surface near the embankment core contact with the downstream shell, as depicted by the red circle in Figure 3-15. When the mesh size is reduced, as shown in Figure 3-16, the phreatic surface smooths out and becomes more reasonable.
- Tip 2 – If working with a large, complex model in which a smaller mesh size may not be reasonable, consider reducing the mesh size at discrete seepage control features (e.g., filters/drains, toe drains, relief wells), or other specific regions of interest, to better capture the trends surrounding these narrow/smaller features, which typically have higher permeability values.

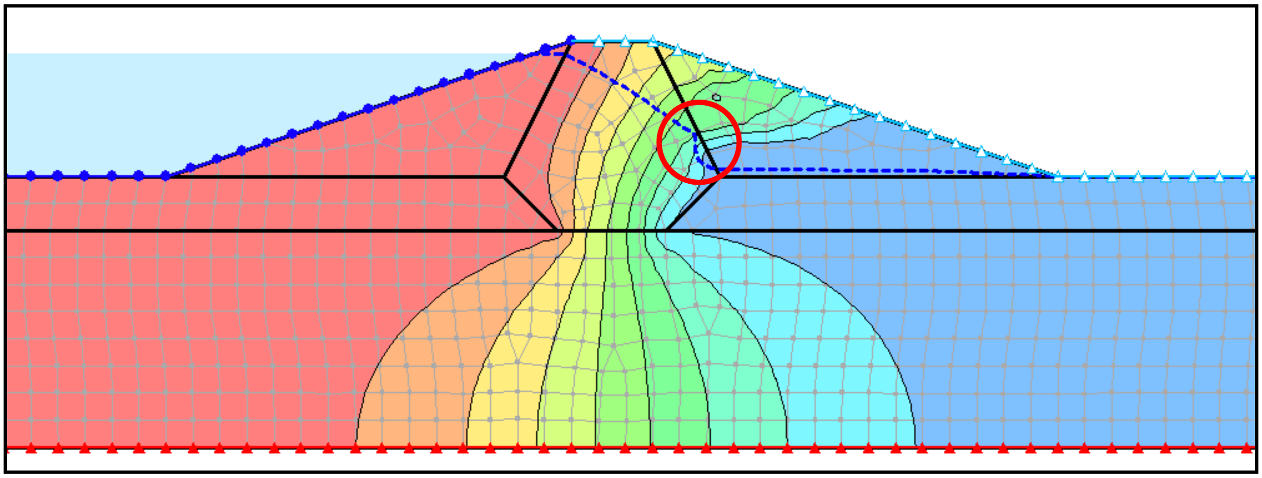


Figure 3-15: Seepage Model Results from the SEEP/W Program Using 10-foot Mesh Size (Approximately 1,000 Elements) (Heitland et al. 2020)

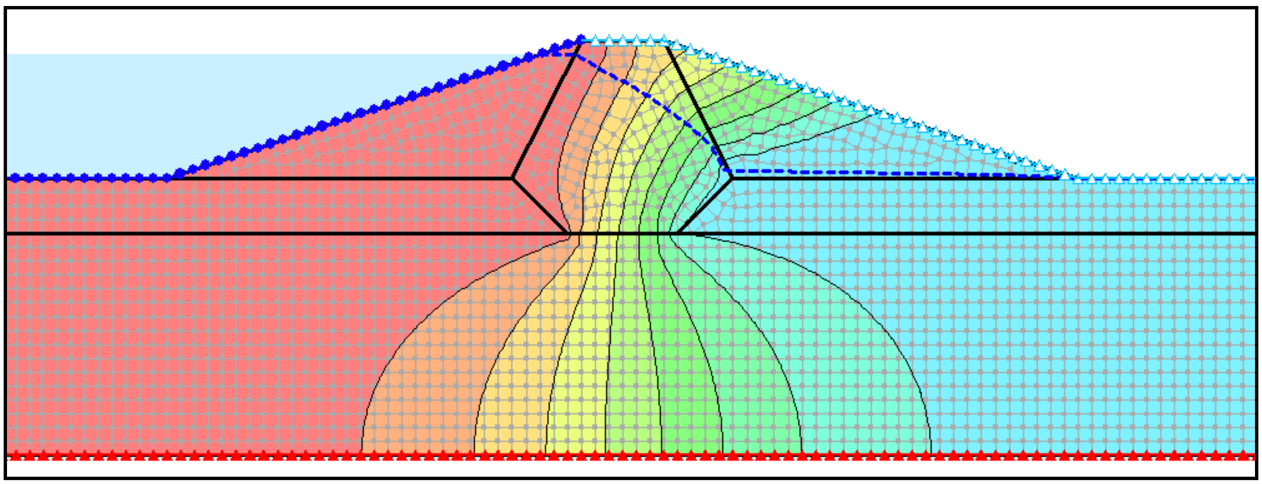


Figure 3-16: Seepage Model Results from the SEEP/W Program Using 5-foot Mesh Size (Approximately 4,000 Elements) (Heitland et al. 2020)

3.4.4 Model Calibration Considerations

Once the model setup is complete, it is important to calibrate the model based on past performance of the dam and piezometer data (if available). Model calibration is an iterative process performed by changing model parameters to match the results with observed or measured conditions. Most commonly, material properties are used as the calibration parameters. Therefore, permeability and anisotropy ratio become the main calibration parameters. In saturated/unsaturated analyses, unsaturated parameters can also be used for calibration. Provided below are tips for consideration.

- Tip 1 – Start with the most representative values of permeability and anisotropy ratio for the materials and perform the simulation. If the results (phreatic surface) deviate from the observed data, change the permeability of the material in the area where the deviation is significant.
- Tip 2 – Change one parameter at a time.

For most cases, the phreatic surface from the model results is in good agreement with measured results when the resulting phreatic surface is within a few feet of the measured results. However, the final decision on whether the variance between the model results and measured conditions is considered acceptable is site specific and dependent on how accurate the model results need to be for the intended purpose. The accuracy of the calibration will be reflective of the accuracy of any subsequent result using the model.

If the past performance and/or piezometer data are unavailable (e.g., if the dam is new), sensitivity analyses can be performed by using higher and lower input parameters to understand the effect of their variation on the model results.

It is important to acknowledge that calibration can be the most time-consuming and costly part of modeling. Thus, it is critical to have a plan in place and communicate with the dam owner and/or regulator before modeling begins. The plan should consider the following:

- Description of how the calibration will proceed (i.e., the piezometer data and associated reservoir levels that will be applied to the model).
- Description of what will be considered reasonable calibration and the amount of time (i.e., cost) it could take to achieve reasonable calibration.
- When efforts to calibrate a model should cease if a reasonable model is not obtained.
- The parameters that will be calibrated.
- How hydraulic conductivity and volumetric water content functions will be estimated.
- Potential problems that could make calibration difficult, such as flow from an abutment.
- Method to keep track of calibration runs.
- The possibility that the model cannot be calibrated. It is important that this be explained to the dam owner or regulator. It is worthwhile to also note the benefits of the model even if it cannot be calibrated.
- The possibility of using sensitivity analyses for situations where calibration is not possible (e.g., new dam construction).

3.4.5 Model Convergence Considerations

Model convergence issues occur when the model cannot obtain a solution for one or more elements within the seepage model during the simulation. The residual error of the solution is higher than the specified value in a non-converged model. Convergence issues are generally experienced when the model geometry and boundary conditions are complex and the soil property functions are highly non-linear. An important consideration to overcome convergence issues is to start with a simple model. There is no one solution that fits all situations of non-convergence. However, the following tips may be helpful:

- Tip 1 – Simplify the model if it makes sense to do so.
- Tip 2 – Increase or decrease the mesh size.
- Tip 3 – Increase the number of simulation iterations.
- Tip 4 – Decrease the non-linearity of the material property functions (e.g., hydraulic conductivity function) if it makes sense to do so.

3.5 Interpreting, Verifying, and Reporting Results

Seepage modeling is complex and requires practice and experience. Models will often run without error and produce professional-looking results but can be invalid. Successful runs should not be confused with accurate results. Interpreting and verifying the results are not always intuitive to the novice user. Vetting errors and understanding the sensitivity of a model to the potential range of each input parameter should be a priority before using the results. This often requires knowledge gained through personal trial and error experience. Consider seeking guidance from experienced engineers and modelers. If not available in-house, program vendors and developers often offer limited technical support for vetting specific problems.

Specific situations may warrant hiring an expert to perform the seepage modeling. It is easier to create models for steady-state conditions than for transient conditions, but most reservoirs fluctuate during normal operation, and piezometric measurements often lag behind actual reservoir levels. Thus, a transient seepage model may be needed. This adds an additional level of complexity to both developing and calibrating a model. Another factor to consider is the time and expense to learn or relearn the modeling program if it is not used frequently. In these cases, consider answering the question of when a seepage analysis is warranted, but then contract with an expert to perform the modeling and interpret and verify the results.

There may also be merit to getting a secondary review from an expert, as some regulatory agencies may not have the modeling experience to catch problems.

The following sections provide tips, tools, and guidance on how to interpret and verify seepage analysis results, as well as report the results.

3.5.1 Interpreting and Verifying Results

After the seepage model is run and results are obtained, it is important to examine the various output features, as these are visual tools that help users understand and verify the model results. Table 3-7 summarizes the key output features (specific to the SEEP/W computer program) and associated tips for checking the validity of the model results. Although the table is specific to the SEEP/W computer program, the output features and concepts are similar among seepage modeling computer programs.

Table 3-7: Key Seepage Model Output Features for Checking the Validity of Results (Heitland et al. 2020)

Output Feature	Description	Tips/Considerations
Total Head Contours (1)	• Depict where total head values (i.e., pressure head plus elevation) are the same. • Indicator of the direction of seepage flow.	• Total head contours should decrease from upstream to downstream. • Farthest upstream contour should be equivalent to the elevation of the reservoir level.
Flow Paths (1)	• Show individual water particles traveling within the flow regime from upstream to downstream under a steady-state condition.	• Flow paths should intersect total head contours at right angles (or at least close to right angles) for homogeneous sections. • Flow paths may cross above the phreatic surface since water can flow from the saturated to the unsaturated zone, and vice versa.

Output Feature	Description	Tips/Considerations
Phreatic Surface	• Transition from positive to negative pore water pressures. • Boundary between saturated and unsaturated flow.	• Phreatic surface should be the line of zero pressure. • Undulations or irregularities in the phreatic surface may be an indication that the mesh size needs to be reduced.
Pore Water Pressure Contours	• Depict where pore water pressure values are the same.	• Pore water pressure contours should become increasingly more positive below the phreatic surface and increasingly more negative above the phreatic surface. • Useful tool for verifying the model is properly computing the saturated and unsaturated zones.
Flow Vectors	• Depict the direction and magnitude of seepage flow.	• Larger vectors (arrows) indicate higher seepage flow velocities. • Smaller vectors indicate lower seepage flow velocities. • Understanding where high and low permeability zones are in the model should help the user judge whether the relative flow direction and velocity magnitude make sense.
Flow Sections (2016 and Newer Versions of SEEP/W Use Subdomain Graphing Tool)	• A defined section of the model in which unit seepage flow quantity across the section is computed.	• Useful tool for evaluating the seepage flow rate through a specific material region of interest.
Horizontal and Vertical Seepage Gradients	• Change in total head (i.e., head loss) over the length of the flow path. • Two visual options: Option 1 defines an area of the model to compute average seepage gradients and Option 2 looks at seepage gradient contours.	• Option 1 is a useful tool for evaluating minimum, maximum, and average horizontal seepage gradients and vertical (exit) gradients through a specific area of interest. • Option 2 is useful as a check to Option 1. Author preference is to evaluate gradients using Option 1 because point anomalies (i.e., gradient spikes) can be more difficult to define using Option 2. As the contour interval decreases, the gradient point anomaly tends to increase with no set end value. Refer also to the example discussion below.

Note:
(1) Total head contours and flow paths can be used to approximate the flow net. Total head contours are equivalent to equipotential lines in a flow net. However, flow paths are not the same as flow lines in a flow net. Thus, the addition of flow paths to the total head contours can only simulate the flow net. In a flow net, the amount of flow between each flow line (referred to as flow channels) must be equivalent. In a seepage model, flow paths can be drawn at any point within the flow regime of the model such that the flow between flow lines will not always be equivalent.

An example seepage model using the SEEP/W computer program is presented in Figure 3-17 and is used here to provide additional guidance on interpreting results. The example model is a zoned earthfill embankment dam consisting of a sandy clay core with relatively low permeability and a core trench that extends to the top of bedrock. The core is supported by upstream and downstream pervious sand shells that are founded on slightly less pervious alluvium.

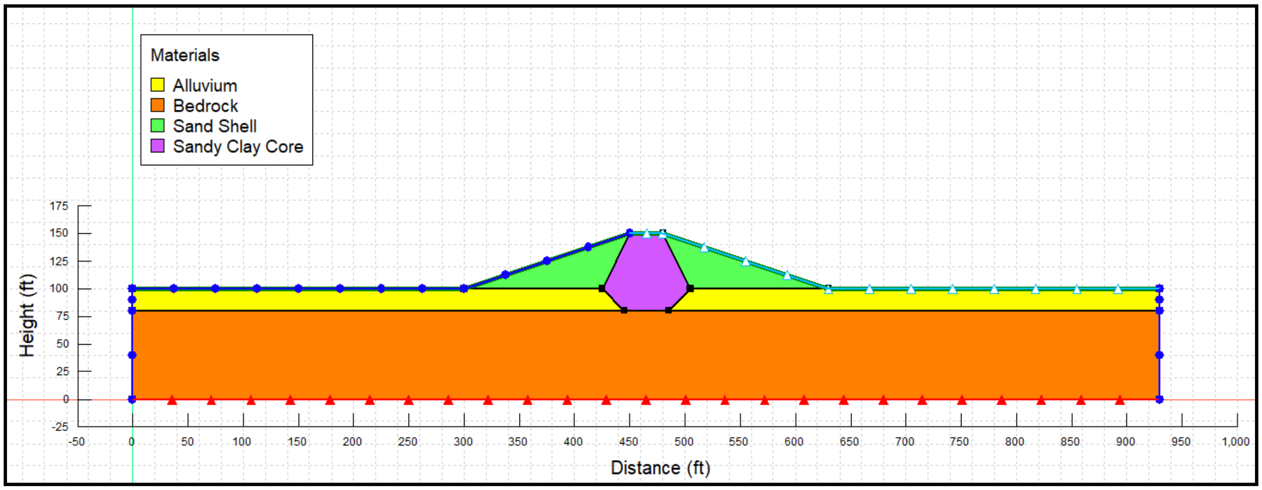


Figure 3-17: Example Seepage Model Using SEEP/W (Heitland et al. 2020)

The validation and interpretation of the seepage model results (shown in Figure 3-18 through Figure 3-25) are summarized as follows:

- Figure 3-18 illustrates the resulting flow net simulation, with the total head contours decreasing from upstream to downstream and the flow paths intersecting the total head contours. As depicted by the red rectangle in Figure 3-18, there are flow paths that cross above the phreatic surface near the downstream embankment toe. This indicates the presence of water in the pore spaces of the shell material near the toe such that the material is not completely unsaturated (or saturated) and is indicative of upward flow.

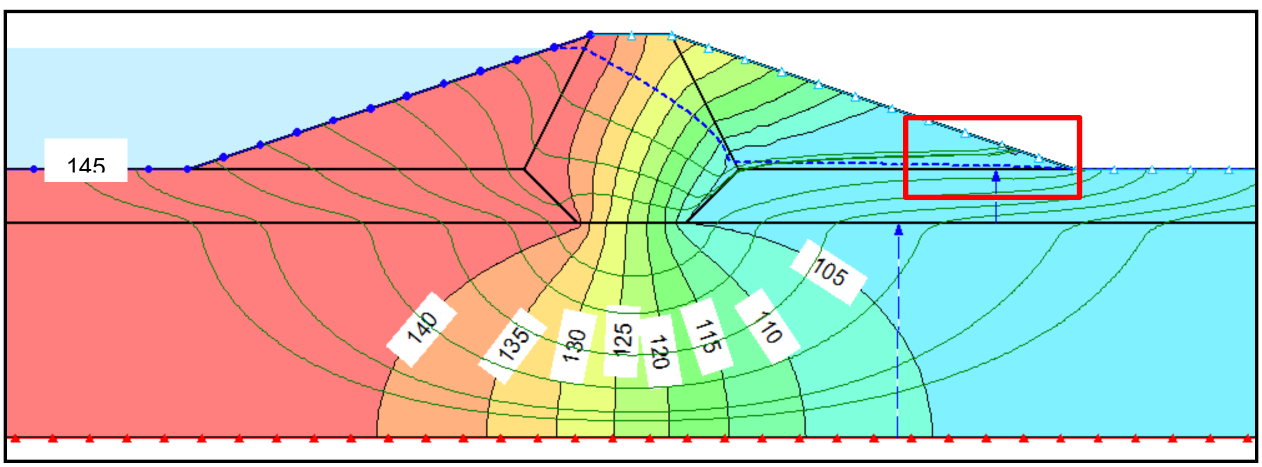


Figure 3-18: Seepage Model Results from the SEEP/W Program Showing Total Head Contours and Flow Paths (Flow Net Simulation) (Heitland et al. 2020)

- Pore water pressure contours are shown in Figure 3-19. The contours become increasingly more positive below the phreatic surface and increasingly more negative above the phreatic surface, with the phreatic surface (dashed blue line) representing the line of zero pressure.

- Review of the phreatic surface shown in Figure 3-18 and Figure 3-19 indicates the embankment core is serving as an adequate seepage barrier, reducing the phreatic surface through the core. Downstream of the core, the phreatic surface extends approximately along the foundation contact to the downstream toe as you would expect with a pervious shell founded on a slightly less permeable foundation.

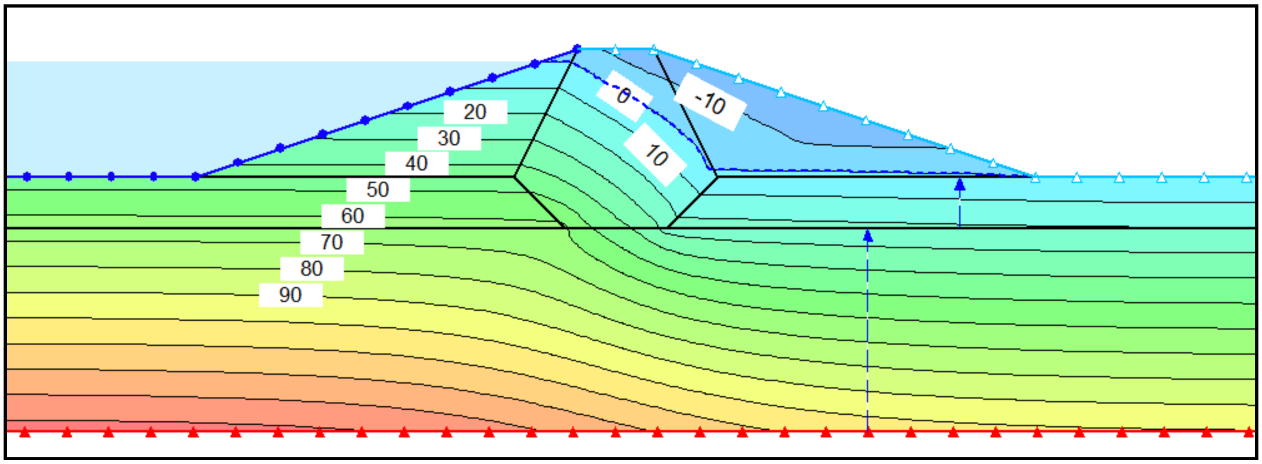


Figure 3-19: Seepage Model Results from the SEEP/W Program Showing Pore Water Pressure Contours (Heitland et al. 2020)

- Flow vectors are shown in Figure 3-20 and indicate that most of the seepage flows below the embankment core trench through the foundation bedrock and then back up through the foundation alluvium. This should be expected because the alluvium has a higher permeability than the bedrock. Based on the size of the flow vectors, the highest seepage flow velocities occur through the alluvium underlying the downstream shell. As seen in the closeup of the red rectangle shown in Figure 3-20, the flow vectors near the downstream embankment toe are oriented slightly upward, indicating upward flow consistent with the flow paths.

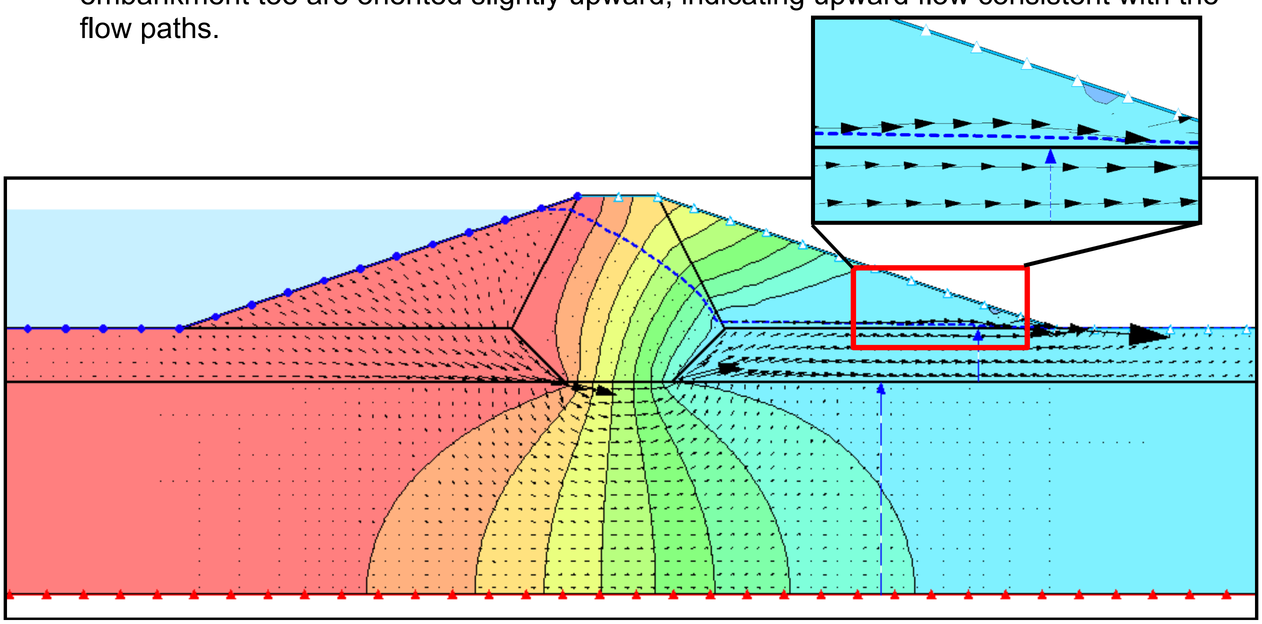


Figure 3-20: Seepage Model Results Showing Flow Vectors (SEEP/W) (Heitland et al. 2020)

- Flow sections through the foundation alluvium and bedrock are shown in Figure 3-21 and Figure 3-22, respectively. In SEEP/W, the graphing tool can be used to evaluate the seepage flow rate at a specified section through a material region by plotting water rate on the vertical axis and the time step on the horizontal axis (which is zero for a steady-state analysis). Review of the resulting seepage flow quantities through the alluvium and bedrock verifies that the location of highest flow is through the alluvium, which is consistent with the flow vectors.

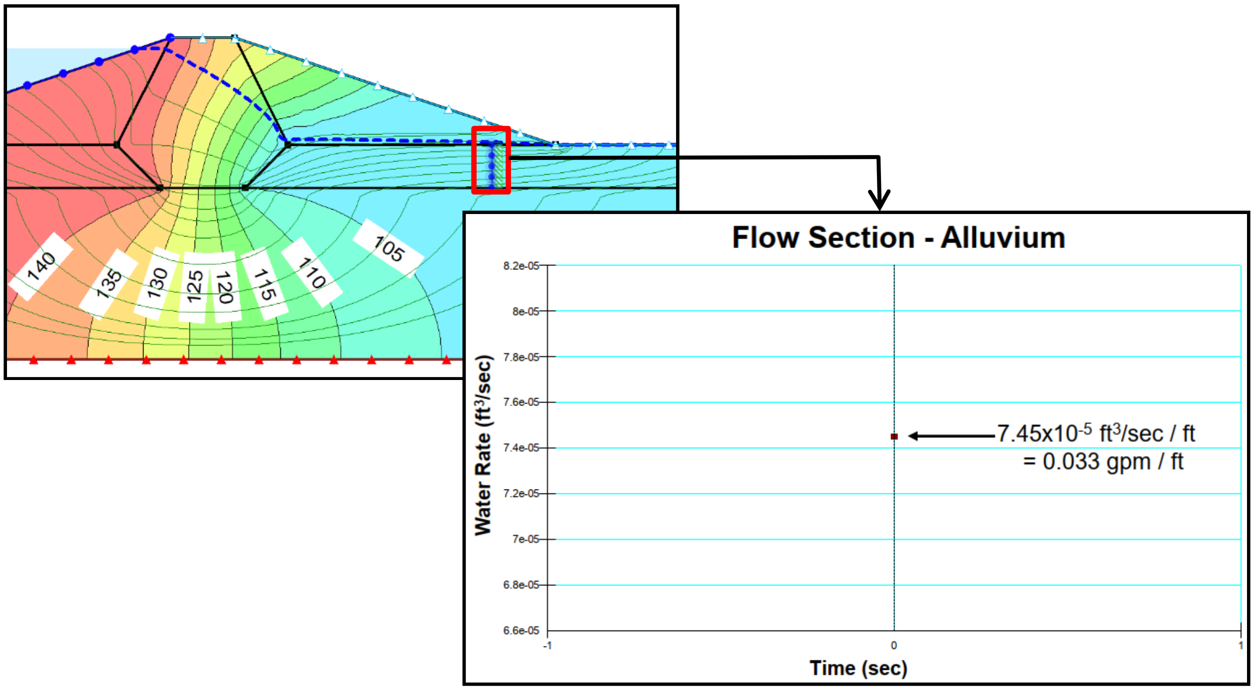


Figure 3-21: Seepage Model Results from the SEEP/W Program Showing Flow Section (Graphing Tool) – Foundation Alluvium (Heitland et al. 2020)

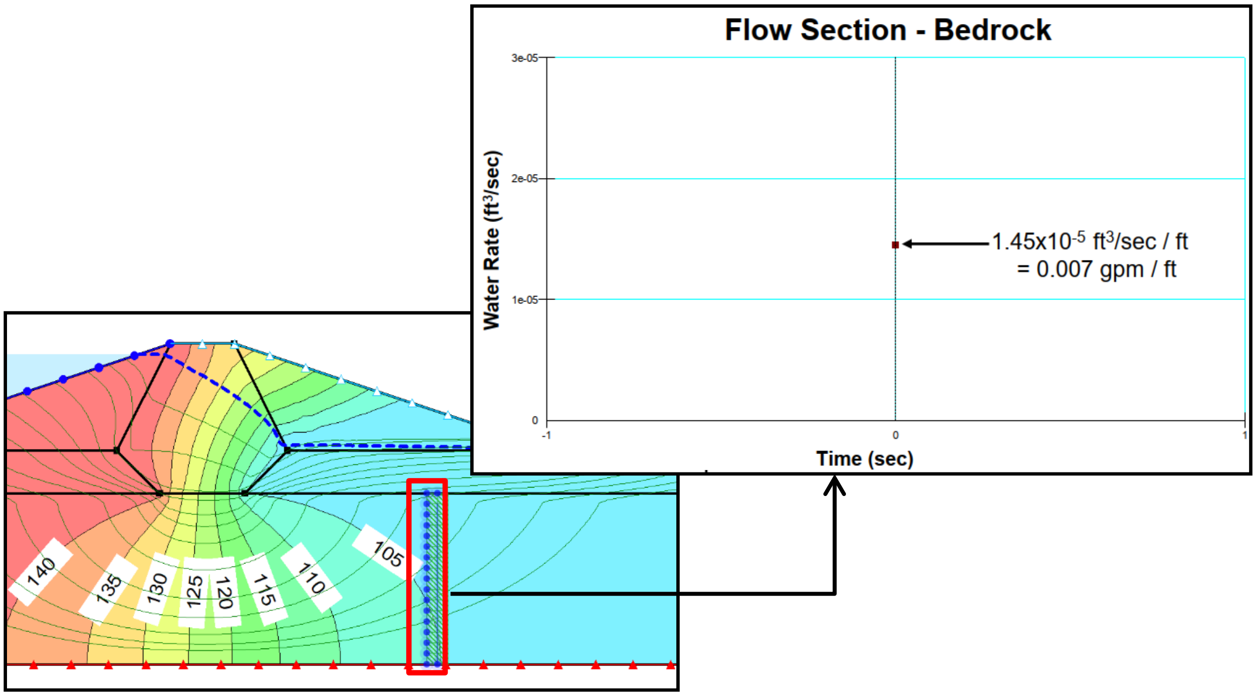


Figure 3-22: Seepage Model Results from the SEEP/W Program Showing Flow Section (Graphing Tool) – Foundation Bedrock (Heitland et al. 2020)

- Horizontal seepage gradients through the downstream foundation alluvium and vertical (exit) seepage gradients at the downstream toe area are shown in Figure 3-23 and Figure 3-24, respectively. Like the flow sections, the graphing tool in SEEP/W can be used to evaluate the seepage gradients at a specified area by plotting gradient (horizontal or vertical) on the vertical axis and the model distance along the specified area on the horizontal axis. The resulting plot can be used to estimate the minimum, maximum, and average gradients through the specified area, which can further be used for evaluating internal erosion potential. As depicted in Figure 3-24, a vertical gradient point anomaly (spike) is identified and a good indication of where to potentially locate a toe drain. In addition to the graphing tool, seepage gradients can be viewed as contours. Figure 3-25 shows the vertical gradient contours at the downstream toe area for comparison to the plot shown in Figure 3-24. The estimated average vertical gradient using the plot shown in Figure 3-24 is consistent with the contour shown at the downstream toe in Figure 3-25. Both figures show the point anomaly representing a potential location for a toe drain. It should be noted that as the contour interval decreases, the vertical gradients at the location of the point anomaly will continue to increase. Thus, it is strongly recommended to evaluate gradients using the graphing tool, as point anomalies can be more difficult to define using only the contours.

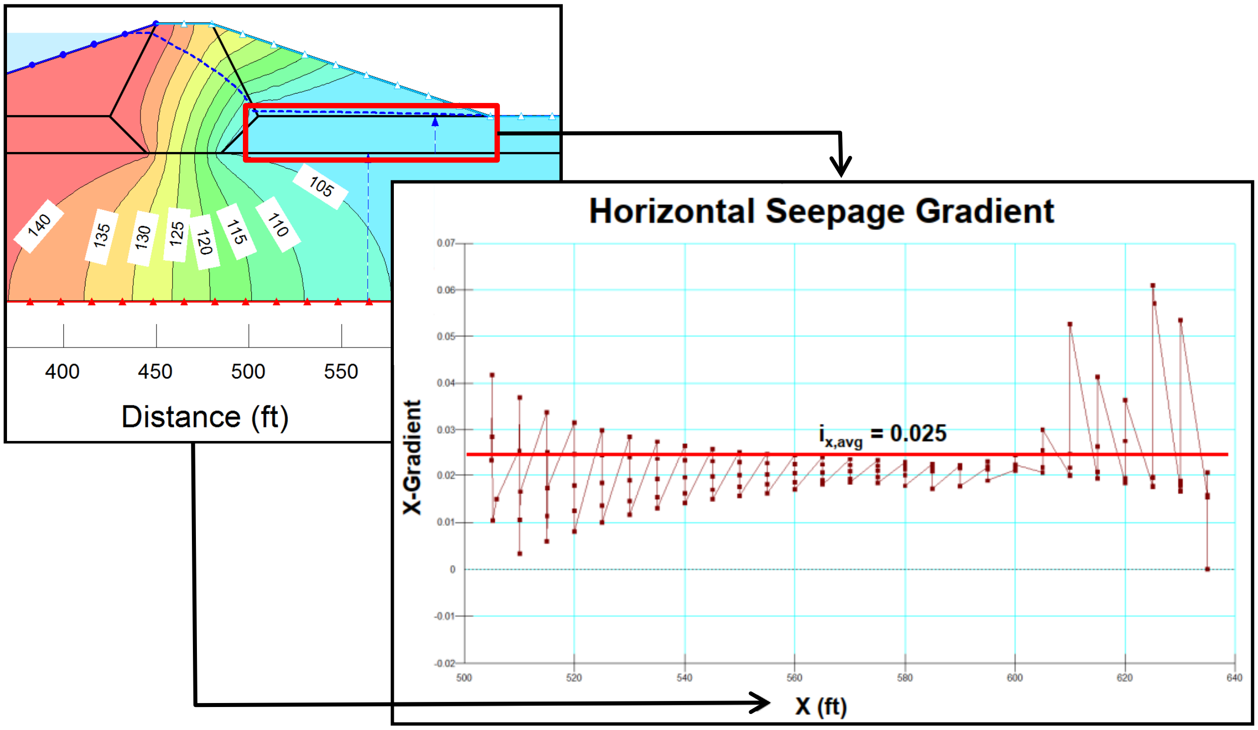


Figure 3-23: Seepage Model Results from the SEEP/W Program Showing Horizontal Seepage Gradients (Graphing Tool) (Heitland et al. 2020)

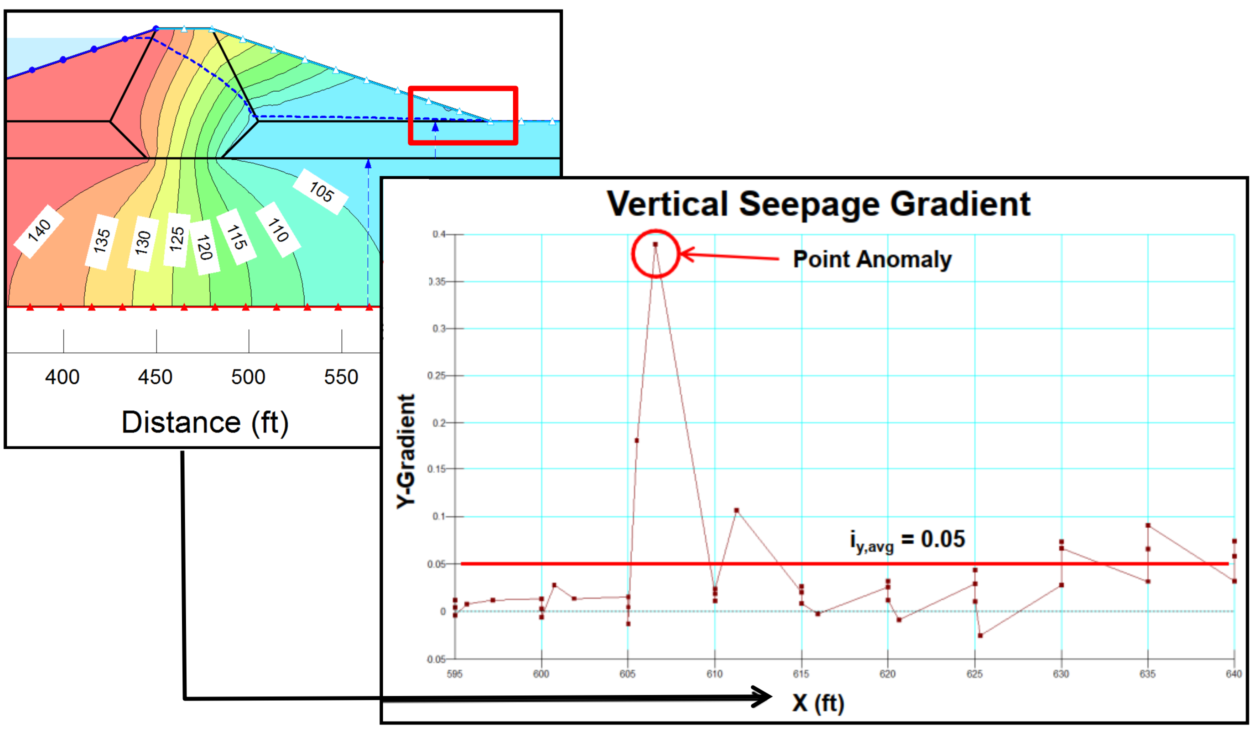


Figure 3-24: Seepage Model Results from the SEEP/W Program Showing Vertical (Exit) Gradients (Graphing Tool) (Heitland et al. 2020)

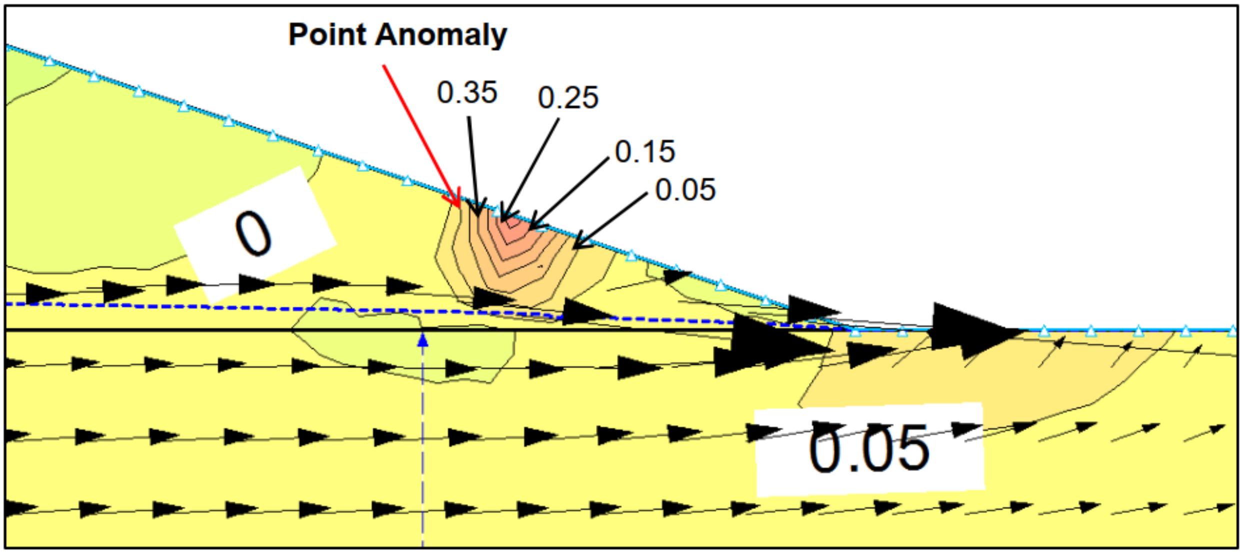


Figure 3-25: Seepage Model Results from the SEEP/W Program Showing Vertical (Exit) Seepage Gradient Contours (Heitland et al. 2020)

For complex seepage models, it is recommended that an experienced engineer with a strong understanding of seepage principles and numerical modeling computer programs perform the seepage analysis and interpret and verify the results. In some cases, well-drawn flow nets of similar designs can be referenced to help verify the general seepage conditions from a model's output.

3.5.2 Reporting Results

Upon completion of a seepage analysis, the analysis should be adequately documented in a report for submittal to the dam owner and/or regulator. The report should include sufficient detail for the reader to understand the purpose of the analysis, how the analysis was performed, the results of the analysis, and recommendations. Seepage analysis reports should include a discussion on the following topics:

- Purpose – Clearly define the objective of the seepage analysis. Refer to Section 3.3.1.
- Methodology – Describe the seepage analysis method and approach (steady-state or transient, 2D or 3D, computer program used, etc.). Refer to Section 3.4.1.
- Model Geometry – Summarize the embankment geometry and internal zoning and the foundation stratigraphy modeled in the seepage analysis. Refer to Section 3.4.2.1.
- Material Properties – Summarize the seepage material properties (e.g., permeability and anisotropy ratio) assigned to the embankment and foundation materials and explain how they were developed. Refer to Section 3.4.2.2.
- Boundary Conditions – Summarize the boundary conditions assigned to the exterior boundaries of the seepage model and their basis. Refer to Section 3.4.3.2.
- Model Calibration – If applicable, describe how the model was calibrated based on past performance of the dam and piezometer data (if available). Refer to Section 3.4.4.
- Results – Provide summary tables and figures of the results and summarize the interpretation of the results. This may also include a discussion on the uncertainties of the

evaluation and the results of sensitivity analyses to evaluate the uncertainties. Refer to Section 3.5.1.

- Recommendations – If applicable, provide recommendations for action (e.g., increased monitoring, reservoir storage restrictions, remediation) or additional investigations based on the results. The results may also be used for alternative design recommendations.

4. Slope Instability of Dams

4.1 What Is Slope Stability?

Slope stability is the ability of a slope to resist the driving forces tending to move earth materials downslope. The slopes of an embankment dam can move downward and outward under the force of gravity. Instability occurs when there is an imbalance of driving and resisting forces. Generally, upstream earth embankment slopes should be no steeper than 3H:1V (horizontal to vertical), and downstream earth embankment slopes no steeper than 2H:1V. The stability of an embankment can be adversely affected by excessive stresses on the crest or slopes, sudden addition or loss of water in the reservoir, changes in internal pore pressures, or loss of materials due to erosion (both internal, such as piping or internal erosion, and external, such as surface erosion). Slides, slumps, slips, cracking, excessive settlement, and other deformations are forms of instability. Figure 4-1 shows an example of slope instability on the downstream slope of an embankment dam.

Figure 4-1: Slope Instability on Downstream Slope of Embankment Dam (FEMA 2016)

All embankment dams in service deform and settle under self-weight and imposed loads. Deformations occur as a response to the weight of a dam and routine operations of the reservoir, including reservoir drawdown and flooding. Excessive deformations occur when movements exceed tolerable limits.

4.2 Slope Instability-Related Issues

Slope instability-related failures of embankment dams are rare, comprising about 6 percent of all dam failures (Foster et al. 2000). Although not as common as seepage-related failures (discussed in Section 2.2), failures due to slope instability can be sudden and catastrophic, often with little to no warning signs. For this reason, it is important to evaluate the factor of safety against slope instability under all expected loading conditions through analysis.

This guidance document discusses the planning and interpretation of slope stability analyses, which can provide insight into slope instability-related issues. Identifying and understanding potential slope instability-related issues prior to conducting a slope stability analysis will ultimately help in establishing the objective of the stability analysis.

For new embankment dams, slope instability can occur during construction by exceeding the strength of the foundation or new embankment fill. For existing embankment dams, slope instability is typically triggered by some change in condition, such as the occurrence of a rapid drawdown, flood, or seismic event, which can increase the driving forces and decrease the resisting forces. A change in seepage or settlement over time can also result in instability. Figure 4-2 illustrates various types of slope instability failures of embankment dams.

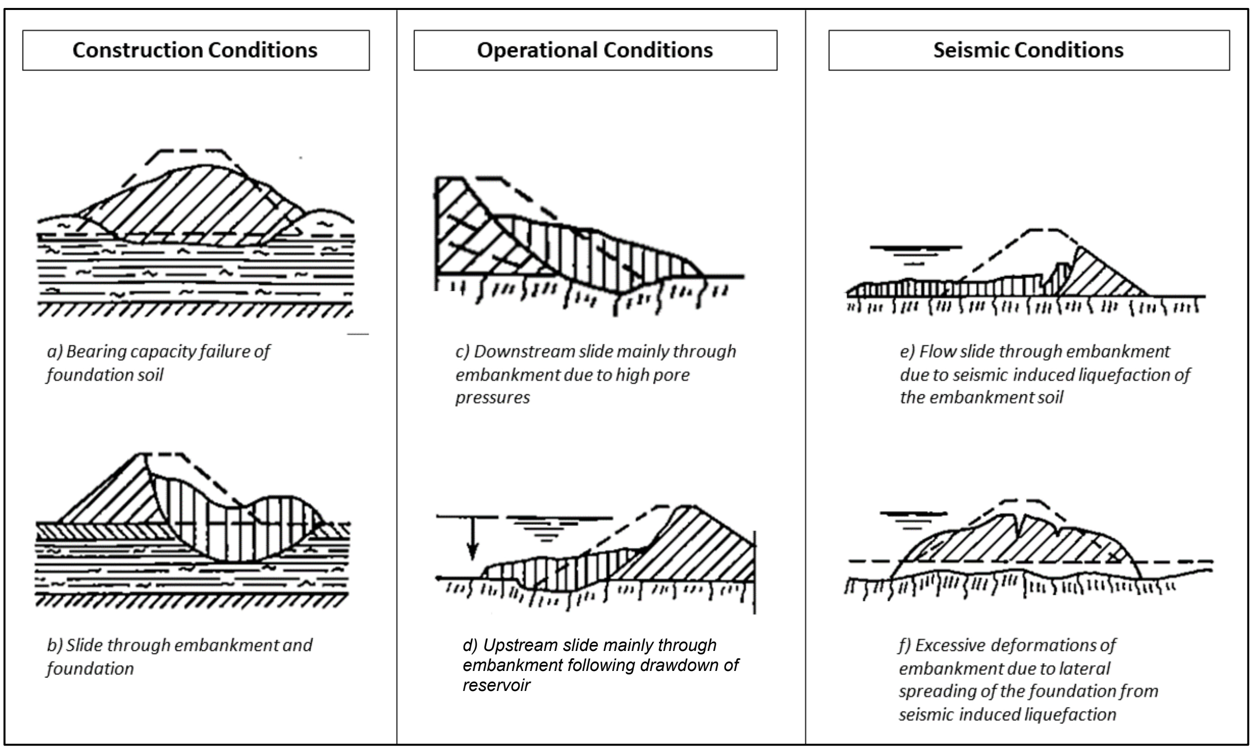


Figure 4-2: Types of Slope Instability Failures of Embankment Dams (Reclamation 1988)

Slope instability failures under construction or operational conditions are often the result of design deficiencies, poor construction, and/or neglected remediation actions. Under seismic conditions, slope instability failures are often the result of seismic-induced deformations and/or strength loss caused by liquefaction (sand-like soils) or cyclic softening (clay-like soils). Excessive deformations may lead to the dam being overtopped, or differential settlement may cause transverse cracks allowing rapid flow of water through the embankment, resulting in dam failure due to internal erosion or elevated pore pressures.

Regular visual inspection is the best tool for dam owners and engineers to assess the safety of an embankment dam when it comes to both seepage- and slope instability-related issues. Visual observations or indicators related to potential seepage-related issues are discussed in Section 2.2. Visual indicators related to potential slope instability-related issues may include the following (AECOM 2013):

- Longitudinal cracks on the embankment crest or slopes – Figure 4-3.
- Wet areas or seepage on the downstream embankment slope or toe, indicating an adverse internal phreatic surface within the embankment – Figure 4-4. The relationship between reservoir level and seepage quantity and quality should be established and used to compare successive observations.
- An apparent embankment slope failure, slump, or scarp – Figure 4-5.
- Erosion or sloughing of the downstream embankment slope, which results in oversteepening of the overall slope.
- Bulges at or downstream of the embankment toe.
- Depressions or sinkholes in the embankment crest or slopes.
- Displaced riprap (on the upstream embankment slope), crest station markers, or fence lines, indicating movement.
- Changes in the appearance of the normal reservoir waterline against the upstream embankment slope at multiple water levels.

Figure 4-3: Severe Longitudinal Crack on Downstream Embankment Slope (AECOM 2013)

Figure 4-4: Seepage Exiting Downstream Embankment Slope (AECOM 2013)

Figure 4-5: Slope Failure on Downstream Embankment Slope (AECOM 2013)

5. Slope Stability Analysis of Dams

A slope stability analysis is a computational method that models the stability conditions of an embankment dam and results in a factor of safety against sliding (i.e., slope instability). The stability or instability of a mass of soil depends on its weight, the external forces acting on it, and the shear strengths and pore water pressures along a slip surface, such as that shown in Figure 5-1. A slip surface is an assumed surface along which sliding may occur when the forces causing instability of a soil mass exceed the shear strength of the soil (i.e., shear resistance). If the shear resistance of the soil along the slip surface exceeds that necessary to provide equilibrium, the mass is stable. If the shear resistance is insufficient, the mass is unstable. Thus, the factor of safety against sliding (FS) is expressed by the equation:

$$FS = \frac{S}{\tau}$$

Where: S = Available Shear Strength
τ = Equilibrium Shear Stress

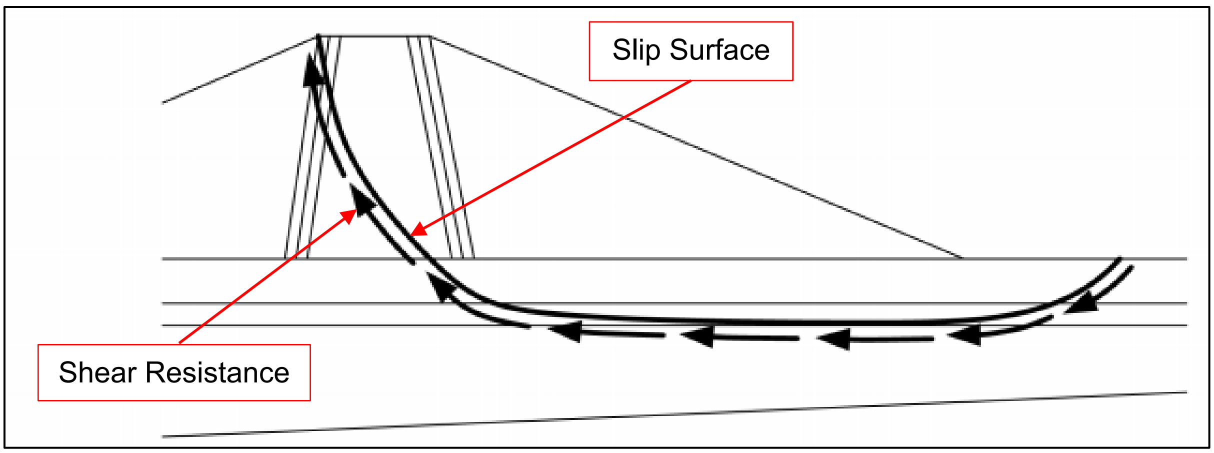


Figure 5-1: Embankment Slope and Potential Slip Surface (USACE 2003)

The most commonly used and accepted slope stability analysis methods for evaluating embankment dams are limit equilibrium methods, which are often implemented in computer programs.

The results of a slope stability analysis can be used for the following:

- Verifying slopes meet minimum factor of safety criteria established by federal and/or state regulatory agencies,
- Evaluating piezometer thresholds to maintain stability, and
- Designing slope stabilization measures (e.g., berm, buttress, slope flattening).

5.1 Slope Stability Analysis Principles

The theoretical principles that govern the equilibrium of a soil mass and that are used to understand and evaluate the slope stability of embankment dams are based on soil mechanics. The concepts of stress-strain behavior, undrained and drained conditions, and total and effective stresses are of fundamental importance in the mechanical behavior of soils. This section provides a basic overview of the principles of soil mechanics for slope stability analysis and was primarily adapted from AECOM (2015) and France et al. (2015). Selection of appropriate shear strengths for stability analysis is presented in Section 5.4.1.2. For more information on slope stability analysis principles beyond that discussed below, refer to additional references in Section 7 (Duncan et al. 2014; Reclamation 2011; USACE 2003).

5.1.1 Stress-Strain Behavior

Understanding the stress-strain behavior of different soils is fundamental to understanding how to characterize and test shear strength. Soils do not generally exhibit significant tensile shear strength; they fail in shear under compression or extension loading. Soil shear strength depends on the following:

- Types of soil particles and mineralogy,
- Consolidation pressure,
- Drainage allowed,
- Stress history (including overconsolidation), and
- Stress paths.

For slope stability analysis, the assumption is usually made that the stress-strain curves of all soils involved reach a maximum shear stress, which then remains constant with further strains, as shown by the generalized stress-strain curve in Figure 5-2. If a material shows minor change in shear stress with increasing shear strain or deformation, that material is said to have a ductile behavior and is not sensitive to deformation. However, if a material shows lower shear stresses with increased deformation, that material is called brittle or sensitive because of its softening behavior under strain. For a brittle or sensitive material, the peak shear strength versus the post-peak shear strength is compared and discussed as peak-post-peak behavior.

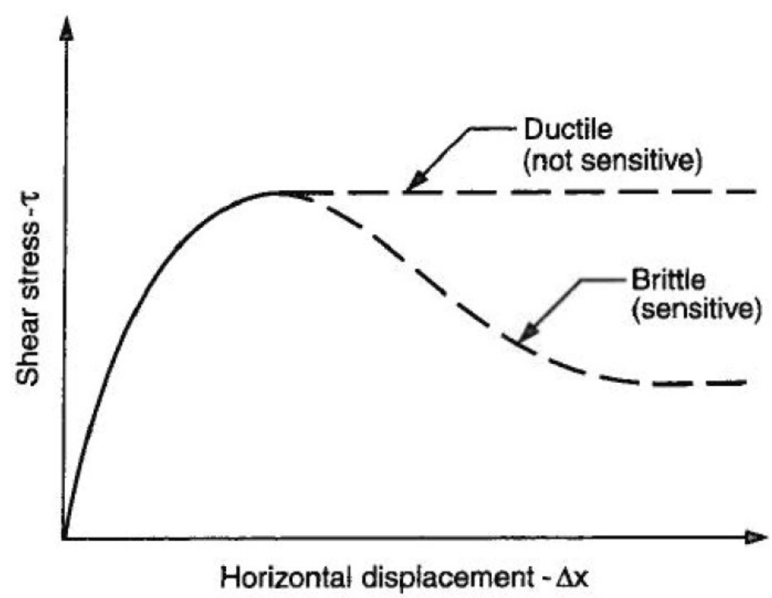


Figure 5-2: Generalized Stress-Strain Curve (Duncan et al. 2014)

The primary factors controlling the shape of the stress-strain curve of soils include the following:

- Soil type,
- Initial structure and state of particle arrangement, and
- Method of loading.

5.1.2 Undrained and Drained Conditions

When saturated or partially saturated soils are loaded in shear, they tend to change in volume. Loose sands or normally consolidated clays tend to decrease in volume (contract), while dense sands or overconsolidated clays tend to increase in volume (dilate). If the loading is applied slowly enough, pore water will flow into or out of the soil mass, the volume of the soil mass will change, and pore water pressures will not change. However, if the loading is applied more quickly than drainage can occur, pore water pressures will be generated within the soil mass. Positive pore pressures will generate in loose sands or normally consolidated clays due to the tendency to compress, while negative pore pressures will generate in dense sands or overconsolidated clays due to the tendency to expand. Coarse-grained soils (sands and gravels) have relatively high permeabilities and sufficient drainage capacity to prevent pore water pressures from changing for most loadings (with the exception of seismic loading, which is beyond the scope of this guidance document), while fine-grained soils (clays and silts) have low permeabilities and can develop excess pore water pressures during some static loading conditions.

Undrained conditions occur when loading is applied more rapidly than soil can drain. Under undrained conditions, water cannot flow into or out of the soil in the length of time the loading is applied. As a result, pore water pressures increase or decrease in response to changes in load, as described above. Drained conditions occur when loading is applied slowly enough relative to the permeability of the soil that pore water can drain. Pore water pressures do not change under drained loading conditions because water can move into or out of the soil freely in response to changes in load.

Hence, whether a particular loading should be considered undrained or drained is dependent on rate of loading, soil permeability, and the distance over which drainage must occur to prevent pore water pressure changes. Duncan et al. (1990) provide a logical basis for estimating the degree of drainage to evaluate whether a material will behave in a drained or undrained manner during rapid drawdown. This basis can be extended to other possible loading conditions to evaluate whether the loading would be drained or undrained by using the dimensionless time factor (T), which is expressed by the equation:

$$T = C_v t / D^2$$

Where: C_v = Coefficient of Consolidation
t = Loading Time
D = Length of Drainage Path

Table 5-1 summarizes typical ranges of coefficient of consolidation values for various soil types.

Table 5-1: Typical Coefficient of Consolidation Ranges by Soil Type (Duncan et al. 1990)

Soil Type	Coefficient of Consolidation, C_v (ft^2/day)
Coarse Sand	>10,000
Fine Sand	100 to 10,000
Silty Sand	10 to 1,000
Silt	0.5 to 100
Compacted Clay	0.05 to 5
Soft Clay	<0.2

If the value of T exceeds 3.0, it is reasonable to treat the material as drained. If T is less than 0.01, it is reasonable to treat the material as undrained. If T is between these two limits, both possibilities should be considered. Alternatively, soils having a permeability greater than approximately $1x10^{-3}$ centimeters per second (cm/s) can be considered to be free-draining under static loading, as a general rule of thumb (Duncan et al. 2014). Although conditions can be intermediate between undrained and drained, loading conditions are almost always modeled as either one or the other. In some cases, when it is not clear whether the loading conditions are undrained or drained, both cases are considered in the analysis.

5.1.3 Total and Effective Stresses

Total stresses within a soil mass include both stresses resulting from forces transmitted through inter-particle contacts and pore water pressures. Effective stresses within a soil mass include only stresses resulting from the forces transmitted through inter-particle contacts. At any given location, the effective stress equals the total stress minus the pore water pressure.

Soil shear strengths can be defined as a function of either total stresses or effective stresses. Effective stress methods should always be used for drained loading conditions. For undrained loading, the analyst needs to choose between total stress and effective stress methods. Total stress methods are used when it is easier to predict the shear strength during undrained loading than it is to predict the pore water pressures during undrained loading, which is almost always the case.

Soil shear strengths are always governed by effective stresses, or inter-particle forces, regardless of loading condition. Total stress strength characterizations are simply used in those cases where pore water pressure responses cannot be easily predicted, and the undrained strength can be more easily predicted. The pore water pressure is implicit in the selected total stress strength; the pore pressure is whatever value is necessary to produce an effective stress state that results in the predicted strength.

5.2 Slope Stability Analysis Methods

The most commonly used and accepted slope stability analysis methods for evaluating embankment dams are limit equilibrium methods, which are often implemented in computer programs. Other slope stability analysis methods include the finite element method and simple slope stability charts. Robust computing capabilities have made the use of stability modeling with complex geometries and shear strength characterizations the current standard of practice. This section provides a basic overview of the typical slope stability analysis methods, focusing specifically on limit equilibrium methods.

5.2.1 Limit Equilibrium Methods

Limit equilibrium methods of analysis is the oldest form of numerical slope stability evaluation. The history of slope stability analyses is described by GEO-SLOPE (2012) starting in 1936, when Fellenius introduced the Ordinary or Swedish method of slices. In the mid-1950s, Janbu and Bishop developed advances in this method. The advent of computers in the 1960s led to more rigorous methods, such as those developed by Morgenstern and Price and Spencer. One of the reasons limit equilibrium methods were adopted so readily is that solutions could be obtained by hand calculations. The introduction of powerful, personal computers in the early 1980s led to the development of commercial computer programs based on limit equilibrium methods.

5.2.1.1 General

Limit equilibrium methods use one or more of the equations of static equilibrium applied to the soil mass bounded below by an assumed slip surface and above by the surface of the slope. The three equations of static equilibrium include: (1) equilibrium of forces in the vertical direction ($\sum F_y = 0$), (2) equilibrium of forces in the horizontal direction ($\sum F_x = 0$), and (3) equilibrium of moments about any point ($\sum M = 0$). Two different procedures can be used to satisfy static equilibrium: a single free-body procedure or a series of individual vertical slices making up the total free body (termed the procedure of slices). The single free-body procedure is relatively simple to use and includes infinite slope, logarithmic spiral, and Swedish slip circle methods. This subsection focuses on the procedure of slices, which is the most common approach used for the stability analysis of embankment dams.

In the procedure of slices, the soil mass above the slip surface is subdivided into a finite number of vertical slices. The number of slices depends on the slope geometry and soil profile. Figure 5-3 is a schematic illustrating the procedure of slices and typical forces acting on an individual slice.

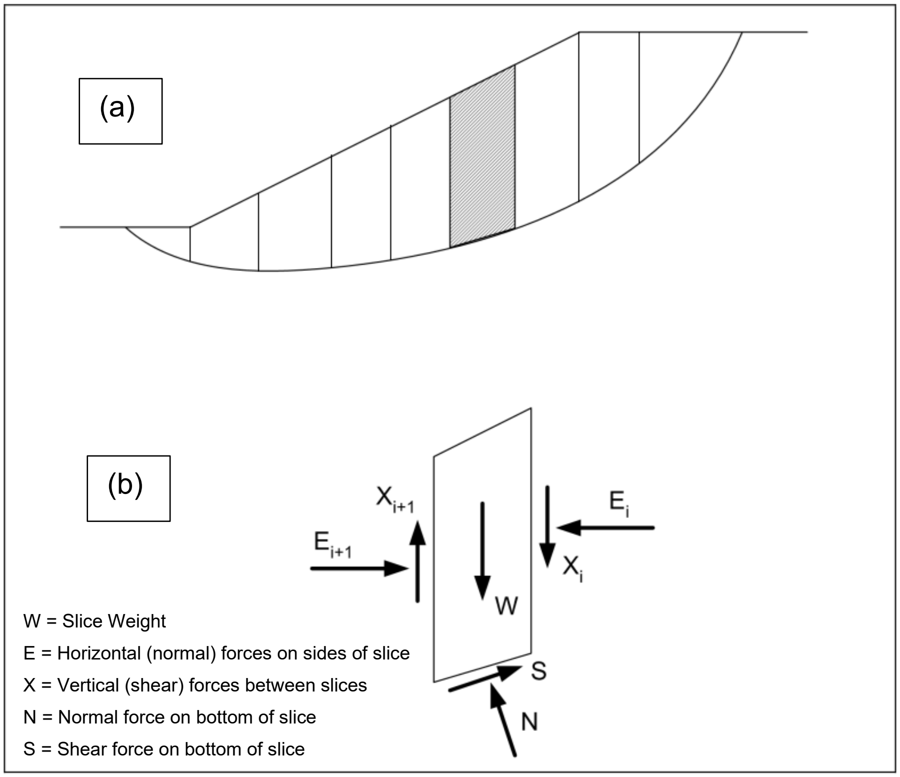


Figure 5-3: Schematic of (a) Procedure of Slices and (b) Typical Forces on an Individual Slice (USACE 2003)

Except for the weight of the slice, all the forces, locations of the forces, and factor of safety are unknowns and must be calculated in a way that satisfies static equilibrium. There are more unknowns than the number of equilibrium equations. Therefore, assumptions must be made to achieve a statically determinate solution.

Several limit equilibrium methods using the procedure of slices have been developed over time and include the following:

- Ordinary Method of Slices
- Simplified Bishop Method
- Modified Swedish Method
- Simplified Janbu Method
- Lowe-Karafiath Method
- Morgenstern-Price Method
- Spencer's Method

Each method subscribes to a different set of assumptions to achieve a balance of equations and unknowns and satisfy static equilibrium. Each method also differs with regard to which equilibrium equations are satisfied. For example, the Ordinary Method of Slices, Simplified Bishop, Modified Swedish, Simplified Janbu, and Lowe-Karafiath Methods do not satisfy all static equilibrium equations. The Ordinary Method of Slices only satisfies overall moment equilibrium about the center of the circle. The Simplified Bishop Method satisfies vertical force equilibrium for each slice as well as overall moment equilibrium about the center of the circle.

The Modified Swedish, Simplified Janbu, and Lowe-Karafiath Methods are “force equilibrium” procedures that satisfy vertical and horizontal force equilibrium for each slice but ignore moment equilibrium. Conversely, the Morgenstern-Price Method and Spencer’s Method satisfy all static equilibrium equations. Methods that satisfy static equilibrium fully are referred to as “complete” equilibrium methods.

Complete equilibrium methods have generally been more accurate than those that do not satisfy complete static equilibrium and therefore are preferable to “incomplete” methods. However, the incomplete methods are often sufficiently accurate and useful for many practical applications, including hand calculations and preliminary analyses.

Primary limitations of the limit equilibrium methods include the following:

- The factor of safety is assumed to be constant along the slip surface. Although the factor of safety may not in fact be the same at all points on the slip surface, the average factor of safety computed by assuming that the value is constant provides a valid measure of stability for slopes in ductile (nonbrittle) soils. For slopes in brittle soils, the factor of safety computed assuming the value is the same at all points on the slip surface may be higher than the actual value.
- The stress-strain behavior of soils is not explicitly accounted for. If the shear strength is fully mobilized at any point on the slip surface, the soil fails locally. If the soil has brittle stress-strain characteristics so that the strength drops once the peak strength is mobilized, the stress at that point of failure is reduced, and stresses are transferred to adjacent points, which in turn may then fail. In extreme cases, this may lead to progressive failure and collapse of the slope. If soils possess brittle stress-strain characteristics with relatively low residual shear strengths compared to the peak strengths, reduced strengths and/or higher factors of safety may be required for stability.
- The initial stress distribution within the slope is not explicitly accounted for. Limit equilibrium methods aim to provide static equilibrium for each slice and make the factor of safety the same for each slice. These inherent concepts and assumptions mean that it is not always possible to obtain realistic stress distributions along the slip surface or within the potential sliding mass.

For more information on the various limit equilibrium methods, refer to additional references in Section 7 (Duncan et al. 2014; GEO-SLOPE 2012; USACE 2003).

5.2.1.2 Selection of Method

A comparison of the slope stability limit equilibrium analysis methods using the procedure of slices is summarized in Table 5-2 and can be helpful in selecting a suitable method for stability analysis. As discussed above, some limit equilibrium methods satisfy all static equilibrium equations, while others satisfy one or two of the equations. Some methods are restricted to circular slip surfaces, while others can evaluate both circular and noncircular slip surfaces. Some methods are more rigorous and require the aid of a computer program, while others can be used without the aid of a computer program and are convenient for checking results obtained from a computer program. Spencer’s Method is a rigorous method and considered the standard of practice when it comes to detailed evaluations of embankment dams.

Table 5-2: Comparison of Slope Stability Limit Equilibrium Analysis Methods Using Procedure of Slices

Feature	Ordinary Method of Slices	Simplified Bishop Method	Modified Swedish Method	Simplified Janbu Method	Lowe-Karafiath Method	Morgenstern-Price Method	Spencer's Method
Accuracy		X			X	X	X
Satisfies Vertical Force Equilibrium		X	X	X	X	X	X
Satisfies Horizontal Force Equilibrium			X	X	X	X	X
Satisfies Moment Equilibrium	X	X				X	X
Circular Slip Surfaces	X	X	X	X	X	X	X
Noncircular Slip Surfaces			X	X	X	X	X
Suitable for Hand Calculations	X	X	X	X	X		

France and Winckler (2010) evaluated the sensitivity of various limit equilibrium analysis methods on the calculated factor of safety for a critical slip surface using the example slope stability model presented in Figure 5-4. For the four limit equilibrium analysis methods shown in the figure, the variation in the calculated factors of safety between analysis methods is relatively small and within the typical level of accuracy of stability evaluations.

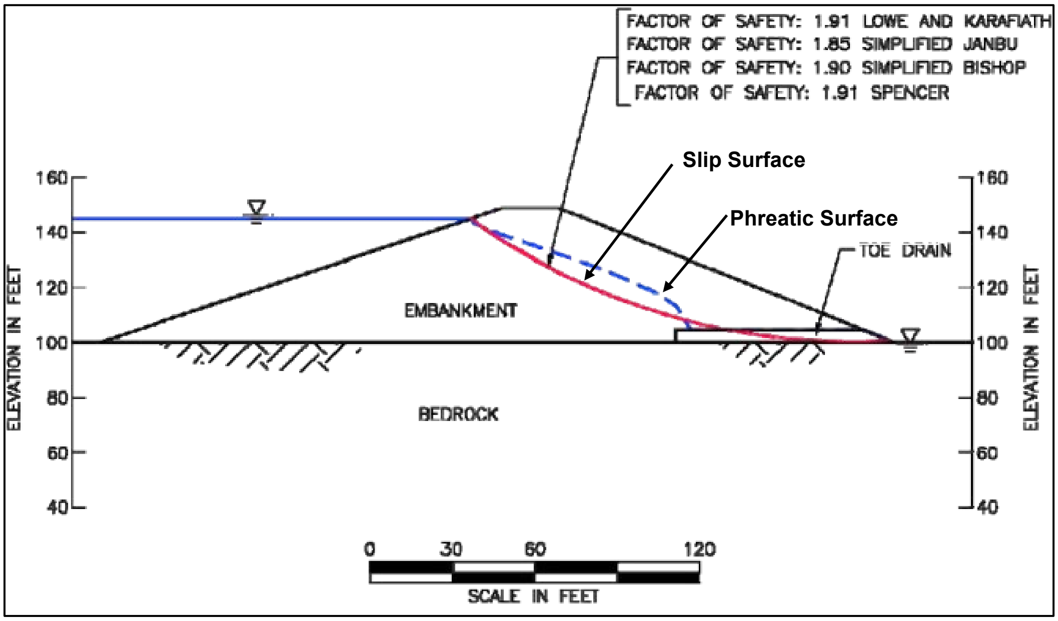


Figure 5-4: Sensitivity of Various Slope Stability Limit Equilibrium Analysis Methods on Factor of Safety (France and Winckler 2010)

5.2.1.3 Computer Programs

Typical computer programs for modeling 2D and 3D limit equilibrium slope stability are summarized in Table 5-3. Computer programs are user friendly and can model a wide variety of slope geometries, soil profiles, soil shear strengths, pore water pressures, and external loads. Most programs also have capabilities for automatically searching for the most critical slip surface with the lowest factor of safety and can handle both circular and noncircular slip surfaces.

Table 5-3: Typical Computer Programs for Modeling Slope Stability Using Limit Equilibrium Methods

Computer Program	Method of Modeling
SLOPE/W (GeoStudio)	2D and 3D • Ordinary Method of Slices • Simplified Bishop Method • Modified Swedish Method • Simplified Janbu Method • Lowe-Karafiath Method • Morgenstern-Price Method • Spencer's Method

Computer Program	Method of Modeling
UTEXAS4	2D • Simplified Bishop Method • Modified Swedish Method • Lowe-Karafiath Method • Spencer's Method
SLIDE	2D and 3D • Ordinary Method of Slices • Simplified Bishop Method • Modified Swedish Method • Simplified Janbu Method • Lowe-Karafiath Method • Spencer's Method
PLAXIS LE	2D and 3D • Over 15 classic procedure of slices methods

In some situations, there may be merit for dam owners or regulators to develop a simplified slope stability model to assist in making decisions. While slope stability models can be valuable in understanding the performance of an embankment dam, the cost of purchasing a computer program can rarely be financially justified for owners and regulators. A new affordable option is GEO-SLOPE's "Basic SLOPE/W," which is a trimmed down version of "SLOPE/W" that is well suited for owner and regulatory needs. Limitations of the basic version include the inability to model a staged rapid drawdown analysis, limited options for soil strength modeling, and the use of only one piezometric line. However, owners and regulators are not generally going to perform a rapid drawdown analysis or use undrained strength functions. Rather, the owner or regulator is likely attempting to understand the embankment and foundation, the drained stability, missing information, and most importantly, whether a more refined stability model may be warranted. The complex stability modeling is often best left to the consulting engineer.

5.2.2 Finite Element Method

While limit equilibrium methods are capable of providing an accurate index of slope stability, the calculated stress distributions are not necessarily representative of actual field conditions. This is because limit equilibrium methods essentially make assumptions to convert a statically indeterminate problem into a statically determinate one. The finite element method can be used to compute the stresses and displacements caused by applied loads. However, it does not provide a value for the overall factor of safety without additional processing of the computed stresses.

The basis for the finite element method is the representation of a body or a structure by an assembly of finite elements. These elements are interconnected at nodal points to form a finite element model. Solutions are obtained in terms of displacements at these nodal points and average stresses in the elements. This procedure can account for various types of stress-strain behavior, heterogeneous conditions, irregular geometry, and complex boundary conditions, as well as time-dependent loading. The computed shear stresses are compared to the corresponding shear strengths to evaluate the factor of safety on an element-by-element basis. This information is used to assess an average factor of safety along an assumed slip surface by taking an average of the calculated factor of safety values for the elements along the shear surface. Similarly, potentially critical shear zones are identified by connecting the elements with low factor of safety values.

In general, slope stability analyses using limit equilibrium methods and the finite element method calculate similar factors of safety for a slip surface. For more information on the finite element method, refer to additional references in Section 7 (GEO-SLOPE 2012; USACE 2003).

5.2.3 Slope Stability Charts

Slope stability charts provide a means for rapid analysis of slope stability by estimating the factor of safety for various types of slopes and soil conditions. Stability charts rely on dimensionless relationships between factor of safety and other parameters that describe the slope geometry, soil shear strengths, and pore water pressures. They are useful for preliminary estimates of stability or checking detailed analyses. However, chart solutions should never be used as the only means of analyzing stability. For more information on example slope stability charts and procedures for using the charts, refer to additional references in Section 7 (Duncan et al. 2014; USACE 2003).

5.3 When Is a Slope Stability Analysis Warranted?

Design of new embankment dams and the more common scenario of reviewing the conditions of existing embankment dams should, as general practice, include evaluating the slope stability of the embankment structure. A slope stability analysis of an embankment dam is a criteria-based evaluation to assess whether the embankment is stable under various loading conditions and meets minimum factor of safety criteria established by federal and/or state regulatory agencies. Because slope stability is a criteria-based evaluation, identifying when a slope stability analysis may be warranted is typically much more straightforward than identifying when a seepage analysis may be warranted. Seepage models can be adjusted based on very little data until the results look like what the modeler hoped they would. Therefore, it takes a more critical assessment when it comes to identifying whether a seepage analysis may be warranted, as discussed in Section 3.3. Assessing whether a slope stability analysis may be warranted is more general and can be related to specific triggers. Triggers that signify a stability analysis will be required may include the following (AECOM 2013; Reclamation 1988):

- Designing a new embankment dam.
- Raising an existing embankment dam.
- Construction of a berm to address stability issues.
- Potential reclassification of a dam to high hazard.
- Deterioration of existing conditions (e.g., oversteepening of the embankment slopes for any reason).
- Visual observations (e.g., longitudinal cracking along the embankment crest or slopes, scarps, toe bulges) indicate that instability may be developing.
- Surface measurement points and/or internal instrumentation (e.g., inclinometers) indicate movement.
- Internal instrumentation (e.g., piezometers) indicate excessive pore water pressures in the embankment and/or foundation.

- Reassurance is needed that a latent, undetected issue has not developed; indicators of such an issue may include embankments with steep slopes (steeper than 2H:1V), soft foundation conditions, high phreatic surface within the embankment and/or foundation, seepage at the slope or toe, observed scarp or bulge, or depression/sinkhole formation.
- Review of design and construction records indicate the presence of previously unrecognized but potentially harmful geologic conditions.
- Gradual loss of strength in foundation clay shales or overconsolidated clays due to swelling.
- Embankment is experiencing an unusually high and perhaps sustained reservoir level.
- Embankment is anticipated to experience an unusually severe drawdown of the reservoir. The severity of drawdown can be in terms of a more rapid rate or to a lower level than it has experienced before.
- Embankment has been exposed to a prolonged dry period followed by rain. The dry period can cause desiccation cracks to develop in some dams; subsequent rain can fill the cracks with water and precipitate slides.
- There is a need for a dynamic deformation analysis.
- There is no slope stability analysis for the embankment dam, stability under specific loading conditions have not been evaluated, or the characterization of specific loading conditions have changed.

In some cases, a slope stability analysis may already exist for an embankment dam, and an assessment must be made as to whether the existing analysis is sufficient in capturing the stability of the current structure. Triggers warranting an additional stability analysis may include the following (Reclamation 1988):

- Existing analysis is not in agreement with the current accepted standard-of-practice methodologies.
- Existing conditions have deteriorated.
- Hazard potential of the dam has increased.
- Embankment has been or will be subjected to loading conditions more severe than designed for.
- Assumed design or analysis parameters cannot be satisfactorily justified.

If satisfactory behavior of an existing embankment dam is observed under loading conditions that are not expected to be exceeded during the life of the structure, then a slope stability analysis may not be warranted. This is provided that adverse changes in the physical condition of the embankment do not occur.

Other considerations when assessing the need for a slope stability analysis should include the following:

- Clearly define the objective of the analysis. Refer to Section 3.3.1 for guidance on developing a clear, well-defined objective. Supplementing objectives with hand drawings can be helpful.
- List the benefits of performing a slope stability analysis and the consequences of not performing an analysis.

It is important to involve dam owners and regulators early in the decision to complete an analysis. What is important to the analyst may vary from that of the owner or regulator. Sometimes the results of an analysis will not justify the cost. The results of the analysis should aim to provide tangible benefits to the project.

If it is determined that a slope stability analysis is warranted, then consult the following sections for guidance for on planning a stability analysis, as well as interpreting, verifying, and reporting the results.

5.4 Planning for a Slope Stability Analysis

Similar to seepage modeling, slope stability modeling can be beneficial in understanding the design and performance of embankment dams. However, a stability model is only as good as the modeler's understanding of the inputs, how the inputs are used in the model, and the modeler's ability to interpret the results. Thus, modelers must be fluent in the model inputs, outputs, and soil mechanics and recognize the model sensitivities to get the most out of a stability model.

This section provides tips, tools, and guidance on planning for a slope stability analysis and focuses specifically on modeling using limit equilibrium methods implemented in computer programs, which are the most commonly used methods for evaluating the stability of embankment dams. Other slope stability analysis methods are introduced in Section 5.2, with references to additional publications for further information on these methods.

Planning for a slope stability analysis involves defining the modeling approach and minimum data requirements. The planning process should also include general modeling considerations. Guidance is provided below for each of these topics. Guidance for interpreting, verifying, and reporting results is provided in Section 5.5.

5.4.1 Modeling Approach

The first step in performing a limit equilibrium slope stability analysis involves defining the modeling approach, which includes considerations for the following:

- Geometry,
- Loading conditions,
- Factor of safety criteria,
- 2D plane strain versus 3D modeling,
- Piezometer threshold analysis, and
- Back analysis.

5.4.1.1 Geometry

Similar to seepage modeling, the geometry of both the dam site and embankment should be taken into consideration in slope stability modeling. In a 2D analysis, cross sections should be selected at locations where critical stability conditions are expected and stability results are required. This will typically include the maximum embankment section, at a minimum, as well as other locations along the dam alignment that may have steep embankment slopes, a narrow embankment crest, or more adverse/variable foundation or embankment conditions. In a 3D analysis, an entire dam can be included in the model.

5.4.1.2 Loading Conditions

Loading conditions are the external loads that apply stresses on an embankment dam. It is important to understand the potential loading conditions for which an embankment dam should be evaluated. Shear strengths in embankment and foundation soils need to be characterized differently for various loading conditions that can occur during the life of the dam. As a result, the slope stability of an embankment dam varies depending on the particular loading condition. A potential sequence of loading conditions during the life of an embankment dam is shown in Figure 5-6.

Typical loading conditions include during-and-end-of-construction, steady-state, rapid drawdown, flood, and post-earthquake. Each of these loading conditions is discussed further below, including selection of appropriate shear strengths. Refer to applicable state or federal agency regulations for specific requirements related to when each loading condition should be evaluated. For more information on slope stability loading conditions beyond that discussed below, refer to additional references in Section 7 (AECOM 2015; Duncan et al. 2014; France et al. 2015; Reclamation 2011; USACE 2003).

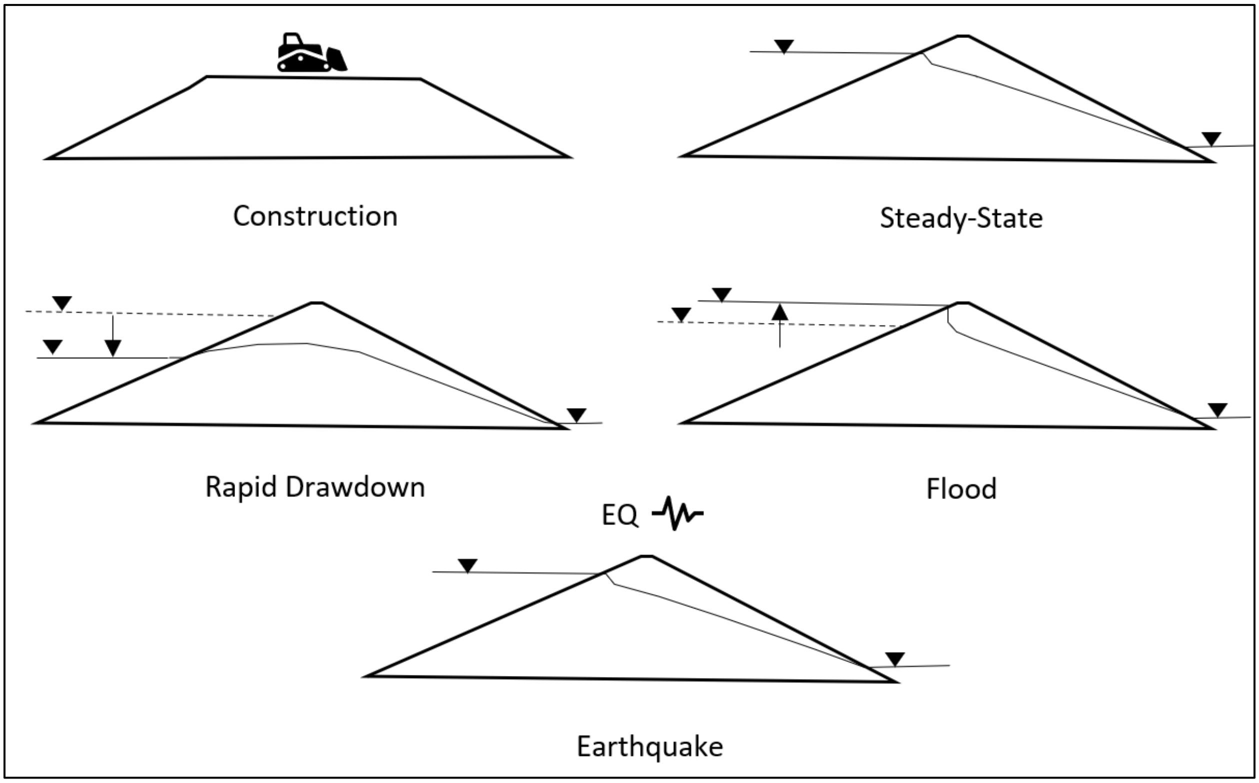


Figure 5-5: Potential Sequence of Loading during Life of Embankment Dam

During-and-End-of-Construction

The during-and-end-of-construction loading condition represents the stability of an embankment dam at specified stages while it is being constructed. Construction may include the initial construction of the dam or additional dam modifications. This loading condition assumes there is no water stored in the reservoir and no phreatic surface present within the embankment during and at the end of construction.

Embankment and foundation materials are evaluated using either drained or unconsolidated-undrained shear strengths depending on the saturation and permeability of the soil. Fine-grained (cohesive) soils generally have low permeability such that little drainage occurs during construction. Coarse-grained (cohesionless) soils have relatively high permeability and are typically free draining. Increased load induced by the placement and compaction of embankment fill during construction may generate excess pore water pressures in the low permeability embankment and foundation materials, thereby developing undrained strength conditions in these materials. Therefore, unconsolidated-undrained shear strengths are assigned to the low permeability materials, and drained shear strengths are assigned to the free-draining materials. The during-and-end-of-construction loading condition should also be evaluated when unconsolidated-undrained shear strengths are estimated to be less than drained shear strengths (contractive soils).

Both the upstream and downstream embankment slopes are evaluated under the during-and-end-of-construction loading condition.

Steady-State

The steady-state (drained) loading condition represents the long-term stability of an embankment dam under normal operating reservoir conditions. This loading condition assumes pore water pressures within the embankment have reached their steady-state seepage condition with no excess pore water pressures remaining from construction, elevated reservoir levels, or other new loading, and all materials are assumed to be fully consolidated under the embankment load. For this loading condition, the phreatic surface and internal piezometric conditions correspond to long-term, normal operating conditions with the reservoir level conservatively modeled at the maximum normal pool level, or service/principal spillway crest elevation. All embankment and foundation materials are assigned drained shear strengths related to effective stresses.

Both the upstream and downstream embankment slopes are typically evaluated under the steady-state loading condition. Often, the factor of safety for the downstream slope is the most critical case. The upstream slope generally has a higher factor of safety than the downstream slope due to the stabilizing effect of water pressure on the upstream slope from reservoir storage, providing a buttressing effect on the embankment. However, the factor of safety of the upstream slope can be low if the slope is very steep or there is something unusual about the embankment zonation or foundation conditions. In this case, analysis of the upstream slope is warranted. If annual pool levels are often at partial pool conditions, a pool level below the maximum normal pool should be evaluated for upstream slope stability, as it may be the more critical case.

Rapid Drawdown

The rapid drawdown loading condition represents the stability of an embankment dam under an assumed instantaneous (i.e., rapid) lowering of the reservoir level from a steady-state pool level, typically taken as the maximum normal pool, to the lowest outlet elevation, removing the buttressing effect of the reservoir on the upstream slope. This loading condition assumes the fine-grained, low permeability embankment and foundation materials below the steady-state phreatic surface are saturated to steady-state conditions prior to drawdown and remain saturated after drawdown. During rapid drawdown of the reservoir, the rate of unloading on the upstream slope may occur rapidly enough that pore water pressures do not have time to dissipate within the saturated, low permeability materials. The excess pore water pressures in these materials therefore develop undrained strength conditions.

The state of practice for evaluating the rapid drawdown loading condition is to use a three-stage slope stability analysis described by Duncan et al. (1990). This method evaluates appropriate shear strength conditions for the materials depending on the stress conditions prior to, during, and after drawdown. The first stage of the analysis calculates the effective stresses for the existing steady-state seepage condition (i.e., steady-state phreatic surface) of the embankment prior to drawdown. In the first stage, the phreatic surface and internal piezometric conditions correspond to long-term, normal operating conditions with the reservoir level conservatively modeled at the maximum normal pool level, and all embankment and foundation materials are assigned drained (effective stress) shear strengths.

The second stage of the analysis calculates the effective stresses for the phreatic surface after drawdown. In the second stage, the phreatic surface is modeled at the lowest outlet elevation, and all saturated, low permeability embankment and foundation materials that cannot drain as the reservoir is lowered are assigned undrained shear strengths based on the effective stresses before drawdown, as calculated in the first stage. Coarser, free-draining materials having a permeability greater than $1x10^{-3}$ cm/s are typically assigned drained shear strengths (Duncan et al. [2014]).

The third stage of the analysis compares the calculated undrained shear strength of the low permeability materials based on the steady-state phreatic surface prior to drawdown (calculated in the first stage) with the calculated drained shear strength of these materials based on the phreatic surface after drawdown (calculated in the second stage). If the undrained shear strength calculated from the first stage is greater than the drained shear strength from the second stage at any point along the slip surface that passes through the low permeability materials, the drained shear strength is used at that location for the third stage analysis. The third stage uses drained shear strengths wherever the undrained shear strength is greater to prevent the analysis from relying on the development of negative pore water pressures for stability. The factor of safety is calculated using the lower of either the undrained or drained strengths for the saturated, low permeability materials from the third stage with the drained strengths for all other materials and reservoir pressures on the upstream embankment slope from the second stage.

Only the upstream embankment slope is evaluated under the rapid drawdown loading condition since this loading condition does not affect the stability of the downstream slope.

Flood

The flood loading condition represents the stability of an embankment dam under a raising of the reservoir level from the steady-state, maximum normal pool level to a flood pool level resulting from a hydrologic or operational event. Inflow produced by hydrologic events can cause the reservoir behind an embankment to rise to levels higher than the normal pool level typically considered for the steady-state loading condition. The higher reservoir level increases the water pressure acting on the upstream slope. Depending on how quickly the reservoir rises and the permeabilities of the embankment and foundation materials, piezometric pressures within the embankment and foundation may increase.

Traditionally, the stability of an embankment dam under the flood loading condition has been analyzed by estimating the steady-state seepage condition that would be expected to develop if the higher reservoir level were in place long enough to allow steady-state seepage to fully develop. The loading condition was evaluated using piezometric pressures estimated for the steady-state seepage condition with the higher reservoir level in conjunction with drained shear strengths assigned to all materials. It was recognized that, in many cases, the permeabilities of the materials involved were low enough that steady-state seepage likely would not develop during flood loading, but the approach was believed to be conservative. However, if this conservative approach with elevated piezometric pressures and drained strengths indicates an unsatisfactory factor of safety, it may be appropriate to consider an alternate approach.

With modern access to computer programs that make transient seepage analyses easier, some analysts have begun to use transient seepage analyses to estimate phreatic conditions expected to develop during the estimated duration of the flood, and then use those seepage conditions in conjunction with drained strengths in the analysis. There are two potential problems with this approach. First, the accuracy of transient seepage analyses is subject to significant uncertainty because of variations in permeability and stratigraphy. The uncertainties in transient analyses are even greater because of additional parameters that must be estimated for the analyses. Second, as was noted in the discussion of the rapid drawdown loading condition, if the flood load is applied more quickly than water can flow in or out of some of the materials, pore water pressures will generate in those materials in response to shear loads, and these shear-induced pore water pressures are not considered in transient seepage analyses. For most large embankment dams, the increase in reservoir level is relatively small compared to the normal reservoir depth, and the changes in shear loads may not be large. But for smaller embankment dams or flood management dams, the increase in reservoir level may be significant, and the undrained loading condition may need to be considered.

Therefore, more recent analysis of the flood loading condition assumes that the duration of the hydrologic event would not be sufficient to develop steady-state seepage in the fine-grained, low-permeability embankment and foundation materials. Pore water pressures within the embankment are initially taken as those developed under steady-state seepage with the reservoir level at the maximum normal pool. The rate of increased loading on the upstream slope resulting from the higher reservoir level during the hydrologic event may occur rapidly enough to generate excess pore water pressures in the saturated, low-permeability materials, thereby developing undrained strength conditions in these materials.

A flood loading condition should therefore be evaluated using a two-stage slope stability analysis to evaluate appropriate shear strength conditions for the materials depending on the stress conditions prior to and during the flood. The two-stage analysis is similar to the first two stages of the rapid drawdown loading condition described above, in which effective stresses for the steady-state seepage condition (i.e., steady-state phreatic surface) of the embankment prior to the flood are calculated in the first stage of the analysis and effective stresses for the phreatic surface during the flood are calculated in the second stage of the analysis. The factor of safety is computed using undrained shear strengths assigned to the saturated, low-permeability embankment and foundation materials based on the effective stresses prior to the flood (calculated in the first stage) and drained shear strengths assigned to the coarser, free-draining materials based on the effective stresses during the flood (calculated in the second stage), as well as reservoir pressures on the upstream embankment slope from the second stage.

Both the upstream and downstream embankment slopes are typically evaluated under the flood loading condition.

Post-Earthquake

The post-earthquake loading condition represents the stability of an embankment dam at the end of shaking resulting from a seismic event. Rapid shaking and accelerations produced by seismic events can induce cyclic loading within the embankment and foundation. This cyclic loading may generate excess pore water pressures in the saturated, fine-grained, low-permeability embankment and foundation materials, thereby developing undrained strength conditions in these materials. Cyclic loading may also cause strength loss (i.e., residual strength conditions) due to liquefaction of saturated, loose, sand-like materials or cyclic softening of saturated, soft, clay-like materials. Thus, this loading condition often requires a liquefaction-triggering and/or cyclic-softening analysis of loose/soft embankment and foundation materials to evaluate the liquefaction or cyclic softening potential of these materials.

Embankment and foundation materials that are not susceptible to liquefaction or cyclic softening can be assigned drained or undrained shear strengths related to steady-state effective stresses prior to the earthquake. Undrained shear strengths are assigned to the saturated, low-permeability materials, and drained shear strengths are assigned to the coarser, free-draining materials. Conversely, embankment and foundation materials that are found to be susceptible to liquefaction or cyclic softening are assigned residual shear strengths. For more information on liquefaction and cyclic softening evaluations, refer to additional references in Section 7 (Idriss and Boulanger 2008).

Both the upstream and downstream embankment slopes are typically evaluated under the post-earthquake loading condition. If the stability analysis under the post-earthquake loading condition indicates an unsatisfactory factor of safety, a numerical dynamic deformation analysis may be warranted to estimate the embankment deformations resulting from an earthquake for comparison against tolerable deformations. Dynamic deformation analyses are highly specialized and beyond the scope of this guidance document.

Further guidance for evaluating whether an embankment dam requires a post-earthquake stability analysis is provided in *Technical Note 5 – Simplified Seismic Analysis Procedure for Montana Dams* (HDR Engineering 2020), which lays out a simplified stepwise path using recent industry publications. Although the ground motions used in this reference are specific to Montana, the procedures are widely applicable.

5.4.1.3 Factor of Safety Criteria

The analyzed stability of a slope is expressed as a factor of safety. A factor of safety greater than 1 indicates the estimated driving forces are less than the resisting forces. However, due to inherent uncertainties in the behavior and characterization of earth materials that compose embankment dams, regulations and good practice require a factor of safety greater than 1 for the various loading conditions. Therefore, it is important to establish factor of safety acceptance criteria for slope stability modeling. Typical factor of safety criteria for slope stability analyses based on Reclamation (2011) and/or USACE (2003) guidelines are summarized in Table 5-4. Criteria may differ among various state and federal agencies.

Table 5-4: Typical Factor of Safety Criteria

Loading Condition		Minimum Factor of Safety
During-or-End-of-Construction		1.3
Steady-State	(Reservoir at maximum normal pool level)	1.5
Rapid Drawdown	Reservoir drawdown from maximum normal pool level to inactive pool (lowest outlet elevation)	1.3
	Reservoir drawdown from inflow design flood (IDF) pool to inactive pool (lowest outlet elevation)	1.1
	Reservoir drawdown from IDF pool to active pool (maximum normal pool level)	1.2
Flood	(Reservoir at IDF pool level)	1.2 / 1.4 (1)
Post-Earthquake		1.2 to 1.3 (2)

Notes:

(1) The Reclamation (2011) and USACE (2003) guidelines differ in their minimum factor of safety for the flood loading condition. Reclamation recommends a minimum factor of safety of 1.2 for the condition in which steady-state seepage develops with the reservoir at the IDF pool level. USACE recommends a minimum factor of safety of 1.4 with the reservoir at the IDF pool level but uses internal pore water pressures taken as those developed under the steady-state seepage condition with the reservoir at the maximum normal pool level. If the IDF pool level is

anticipated to be a relatively small change from the maximum normal pool level in comparison to the dam height and the flood loading duration will not be sufficient to develop steady-state seepage in the low permeability embankment and foundation materials at the surcharge level, the USACE guidelines are considered more representative.

(2) The Reclamation (2011) and USACE (2003) guidelines do not present a minimum factor of safety for the post-earthquake loading condition. A factor of safety of 1.2 to 1.3 is consistent with guidance provided by FEMA (2005) and with standard of practice and should be selected based on consideration of uncertainties. This range of factors of safety is considered to be representative of limited post-earthquake deformations.

5.4.1.4 2D versus 3D Modeling

Historically, slope stability analyses have primarily comprised 2D modeling because of the complexity of 3D modeling and limited computer programs capable of 3D modeling. However, similar to seepage analyses, slope stability analyses have recently been expanding into the 3D realm as the benefits of 3D modeling are becoming clearer and computer programs with 3D capability are becoming more prevalent.

While 3D slope stability modeling can be valuable under certain conditions, 3D modeling is much more rigorous than 2D and therefore requires careful consideration to ensure 3D modeling is the appropriate approach. 3D modeling is more time-consuming, requires a greater level of effort and expertise, and is more costly than 2D modeling. However, 3D slope stability modeling becomes beneficial for conditions in which there are significant 3D cross-valley effects along the embankment dam alignment (e.g., narrow “v”-shaped valley profile) or when evaluating the abutment-embankment contact area (i.e., groin).

5.4.1.5 Piezometer Threshold Analysis

In some cases, it may be beneficial to perform a piezometer threshold analysis to provide a basis for establishing critical levels for piezometers within an embankment dam. A piezometer threshold analysis is conducted by incrementally raising the phreatic surface through the embankment at locations of piezometers until the factor of safety for the critical slip surface is reduced to a target factor of safety. The target factor of safety may be selected to be consistent with the acceptance criteria, or a lower value may be selected to evaluate the critical piezometer threshold levels that may warrant action.

5.4.1.6 Back Analysis

When an embankment slope fails due to instability, a back analysis can be performed to provide useful information into the conditions of the slope at the time of failure. A back analysis is conducted using the resulting slip surface and an assumed factor of safety of 1.0 to back-calculate the unit weights and maximum potential shear strengths of the soils and/or pore water pressure conditions at the time of failure. The back-calculated conditions can be used to inform the design of slope stabilization measures. For some cases, such as with brittle soil, the factor of safety could be significantly less than 1 during the failure. For more information on assessing the conditions of a failed slope through back analysis, refer to additional references in Section 7 (Duncan et al. 2014).

5.4.2 Minimum Data Requirements

Slope stability models require accurate data to obtain reliable results. Understanding the minimum data requirements that go into a stability model is valuable in terms of identifying where data gaps may exist to inform the need for additional investigation and/or studies prior to performing the modeling. The minimum data requirements to develop an efficient and reliable slope stability model for an embankment dam are summarized in Table 5-5, along with typical data sources where the data needs can be found. These minimum data requirements are discussed further below.

Table 5-5: Minimum Data Requirements for Slope Stability Modeling

Data Category	Data Requirements	Typical Data Sources
Model Geometry	• Embankment geometry and internal zoning. • Foundation contact and stratigraphy.	• Topographic or Lidar Surveys • As-Built Construction Drawings • Design Drawings • Design or Construction Reports • Geologic and Geotechnical Investigation Reports
Material Properties	• Unit weights and shear strengths for embankment and foundation materials.	• Geotechnical Investigation and Data Reports • Construction Reports • Published Data
Phreatic Surface	• Pore water pressures in embankment and foundation.	• Piezometer Data / Piezometric Measurements • Seepage Analyses

5.4.2.1 Model Geometry

Slope stability models are typically developed for one or more embankment cross sections along the dam alignment. To develop the model geometry, there must be sufficient available data on the embankment geometry and internal zoning, as well as the foundation contact and stratigraphy. A detailed model geometry will delineate the various embankment zones (core, shells, filters, drains, etc.), foundation layers, and any slope stabilization measures if applicable (berm, buttress, slope flattening, etc.).

The best data source for defining the embankment geometry and internal zoning of an existing embankment dam is typically as-built construction drawings. Ideally, the external geometry (i.e., embankment crest, upstream and downstream slopes, and downstream ground surface) should be defined by a recent topographic or lidar survey. When construction drawings and recent surveys are not available, design drawings and/or design or construction reports can be used for defining the model. However, the modeler should be aware that the as-built and/or current embankment condition may differ from the design condition. Information from geologic and geotechnical investigation reports can also be valuable in verifying the internal zoning and variations in the embankment materials. For a new embankment dam that has yet to be constructed, the embankment geometry and internal zoning are typically defined using design drawings. The design can then be adjusted accordingly based on the results of the slope stability analysis.

The best data source for defining the foundation contact and stratigraphy is typically geologic and geotechnical investigation reports. Identifying and modeling weak seams within the foundation is important because weak seams will typically control the critical slip surface.

5.4.2.2 Material Properties

Slope stability modeling requires assigning material properties to the embankment and foundation materials. These material properties include unit weight and shear strength. Similar to the model geometry, there must be sufficient available data on the embankment and foundation materials to estimate the material properties. Typically, a comprehensive material characterization of the embankment and foundation materials is performed prior to the slope stability analysis to estimate the stability properties using data collected from geotechnical investigations and/or published data. Information on the materials may also be available in construction reports.

There are three types of unit weights: dry unit weight, total (or moist) unit weight, and saturated unit weight. Saturated unit weights are assigned to materials below the phreatic surface, while total unit weights are assigned to materials above the phreatic surface. Often, total unit weights are assigned to all materials in a slope stability model for simplification because stability analyses are not highly sensitive to unit weight, as discussed further in Section 5.4.4.1. Dry unit weights are never assigned to materials in a slope stability model because it is uncommon for soils to be completely dry.

Total and saturated unit weights can be estimated from moisture content and dry unit weight laboratory tests, Proctor compaction laboratory tests, published tables of values, or empirical correlations (typically with blow counts).

Shear strength is the single most influential factor in a slope stability analysis aside from embankment geometry, yet it is also the most complex to characterize. After the loading conditions are selected (Section 5.4.1.2), appropriate shear strengths must be estimated for the embankment and foundation materials. Shear strengths can be estimated from laboratory tests, field tests, and/or various empirical correlations. It is unrealistic to obtain samples that represent the entire range of materials in the field, and soils will behave differently in the laboratory than in the field. Therefore, experience and engineering judgment play a major role in shear strength selection. Good practice is to perform sensitivity analyses for a potential range of shear strengths to compensate for uncertainty.

Characterizing the shear strength of soils is dependent on both the type of soil and whether the soil displays drained or undrained behavior under a particular loading condition. Provided below is a description of the shear strengths typically evaluated for coarse-grained, cohesionless soils (sands and gravels) and fine-grained cohesive soils (clays and silts), including the corresponding laboratory and field testing to measure strengths and empirical correlations to estimate strengths. The description below focuses on the characterization of drained and undrained shear strengths and was adapted from France et al. 2015. For more information on shear strength characterization, refer to additional references in Section 7 (France et al. 2015; USACE 2003). The characterization of residual shear strengths for potentially liquefiable (sand-like) or cyclic softened (clay-like) soils is beyond the scope of this guidance document. For information on the evaluation of residual shear strengths, refer to additional references in Section 7 (Idriss and Boulanger 2008; HDR Engineering 2020).

Coarse-Grained Soils (Sands and Gravels)

Coarse-grained, or granular, soils (sands and gravels) are typically free-draining and defined by drained shear strengths, except for very rapid loading (e.g., seismic loading, which is beyond the scope of this guidance document). Coarse-grained soils have relatively high permeabilities and sufficient drainage capacity to prevent pore water pressures from changing under most loading conditions.

Characterizing the drained shear strength of coarse-grained soils involves evaluating or estimating the effective stress friction angle (ϕ'). The Mohr-Coulomb shear strength envelope for coarse-grained soils goes through the origin of stress, as illustrated in Figure 5-6, and thus the effective stress cohesion (c’) is zero. Coarse-grained soils are therefore also often referred to as cohesionless soils. Although the effective stress cohesion is zero, the strength envelope is often curved, as illustrated in Figure 5-7 for the dense soil example. For mathematical simplicity, the strength envelope may be approximated as linear over the normal stress range of interest for the analysis, which may result in an "apparent" effective stress cohesion, as shown in Figure 5-7. It is important to understand that this is a mathematical convenience, and not a true property of the soil.

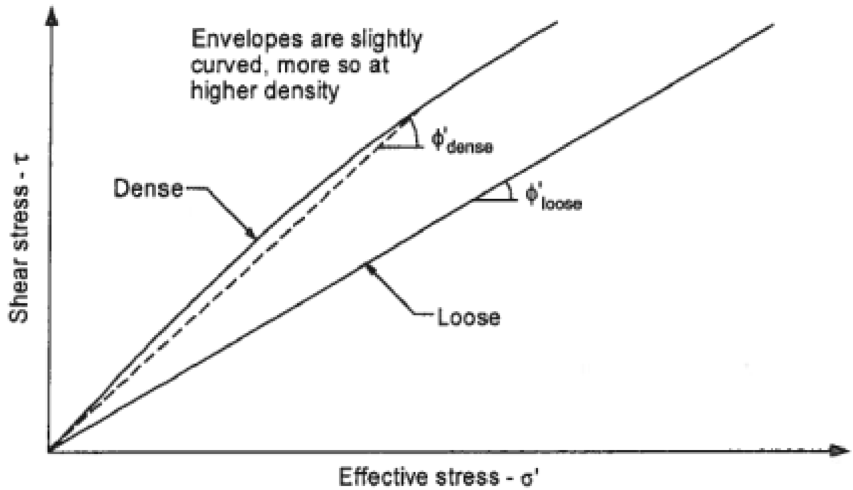


Figure 5-6: Mohr-Coulomb Shear Strength Envelope for Coarse-Grained Soils (Duncan et al. 2014)

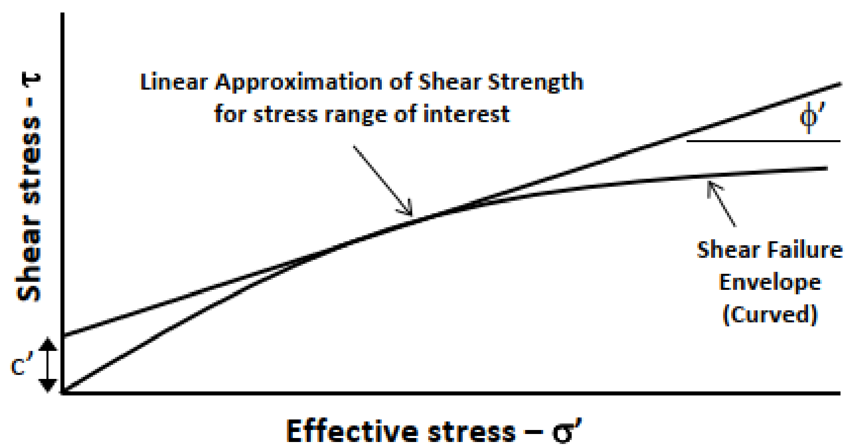


Figure 5-7: Curved Shear Strength Envelope with Linear Interpretation and Apparent Cohesion (c') (AECOM 2015)

Typical factors affecting values of ϕ' for coarse-grained soils include relative density, confining pressure, angularity, and gradation. Values of ϕ' increase as the soil relative density increases, confining pressures decrease, particle angularity increases, and the soil gradation becomes broader (i.e., a wider range of particle sizes are included).

Laboratory tests used to measure values of ϕ' for coarse-grained soils include the consolidated-drained (CD) triaxial shear test, consolidated-undrained (CU') triaxial shear test with pore pressure measurements, and the direct shear (DS) test. It is difficult, however, to obtain undisturbed samples of in-place granular soils or reconstitute the structure of the natural deposits. Laboratory tests are often used to estimate ϕ' for coarse-grained soils that will be placed during construction, such as embankment soils, while empirical correlations are typically used to select strengths for in-place cohesionless soils, such as foundations or existing embankments, as discussed further below. Even when laboratory tests are completed, the results should be checked against expected values based on relative density, gradation, and/or blow count data. The presence of large particles (e.g., scalped samples or rockfill) may make laboratory test results misleading or impractical.

Field tests, including the standard penetration test (SPT), cone penetrometer test (CPT), and shear wave velocity measurements, are often used to estimate values of ϕ' for in-place, coarse-grained soils through the application of empirical correlations. Values of ϕ' for in-place coarse-grained soils can also be estimated using empirical correlations to relative density and confining pressure. Common empirical shear strength correlations are presented in Duncan et al. (2014), which is one of the more comprehensive references for shear strength characterization.

Fine-Grained Soils (Clays and Silts)

Fine-grained soils (clays and silts) are generally defined by undrained shear strengths for short-term loading conditions and drained shear strengths for long-term loading conditions. These soils generally have low permeabilities and can develop excess pore water pressures during some static loading conditions. Given adequate drainage and time, pore water pressures will eventually dissipate in fine-grained soils.

Clays – Characterizing the shear strength of clays is complex and can be quite different for the various loading conditions, as discussed in Section 5.4.1.2. The drained shear strength of clays can be expressed in terms of effective stress (ϕ', c’) strength parameters. The undrained shear strength can be expressed in terms of total stress (ϕ, c) strength parameters or in terms of undrained strength (S_u). Different forms of characterization can be used for S_u, for example, constant S_u, S_u as a function of effective confining stress, and S_u as a function of depth.

The overconsolidation ratio (OCR) of clays also has an impact on shear strength and is defined as the ratio of the maximum pre-consolidation pressure of a soil mass to the current consolidation pressure of that soil mass. For normally consolidated to lightly overconsolidated clays, both undrained and drained shear strengths are of interest. When normally consolidated to lightly overconsolidated clays are loaded in shear, they tend to compress and generate positive pore water pressures (contract), thereby resulting in an undrained shear strength that is less than the drained shear strength. Hence, the lower undrained shear strength must be used when analyzing undrained loading conditions. In contrast, for heavily overconsolidated clays, drained shear strengths are of most interest because these clays tend to expand (dilate) when loaded in shear and therefore generate negative pore water pressures when sheared undrained. The negative pore water pressures result in an undrained shear strength that is greater than the drained shear strength. As discussed in Section 5.4.1.2 with respect to the rapid drawdown loading condition, higher strengths resulting from negative pore water pressures are not normally used in stability analyses because the negative pore water pressures cannot be relied upon in the field. For heavily overconsolidated soils subjected to undrained loading, the drained shear strength parameters are used, practically capping the undrained strength as being no greater than the drained strength.

A typical drained Mohr-Coulomb shear strength envelope for clays is presented in Figure 5-8. For normally consolidated clays, the Mohr-Coulomb strength envelope goes through the origin of stress and the effective stress cohesion (c’) is equal to zero. For overconsolidated clays, the Mohr-Coulomb strength envelope is generally curved in the low stress range but still goes through the origin, such that the effective stress cohesion is equal to zero. Similar to coarse-grained soils, the strength envelope may be approximated as linear over the normal stress range of interest for the analysis, which may result in an “apparent” effective stress cohesion. Again, the strength envelopes with intercepts (shown in Figure 5-8) are a mathematical convenience.

The higher strengths depicted in the lower stress ranges for the overconsolidated clays in Figure 5-8 are the peak strengths for these soils. In most cases, the drained stress-strain behavior for overconsolidated clays exhibits a pronounced peak, followed by a drop to a much lower post-peak, remolded, or fully softened strength. This reduction to post-peak strength occurs at relatively modest strains. The remolded or fully softened strength is the same strength exhibited by a normally consolidated specimen of the same clay. For the reasons cited below for stiff-fissured clay, as well as to guard against progressive failure, it is recommended that for drained loading, the fully softened strength be used for saturated clays in the embankment and foundation.

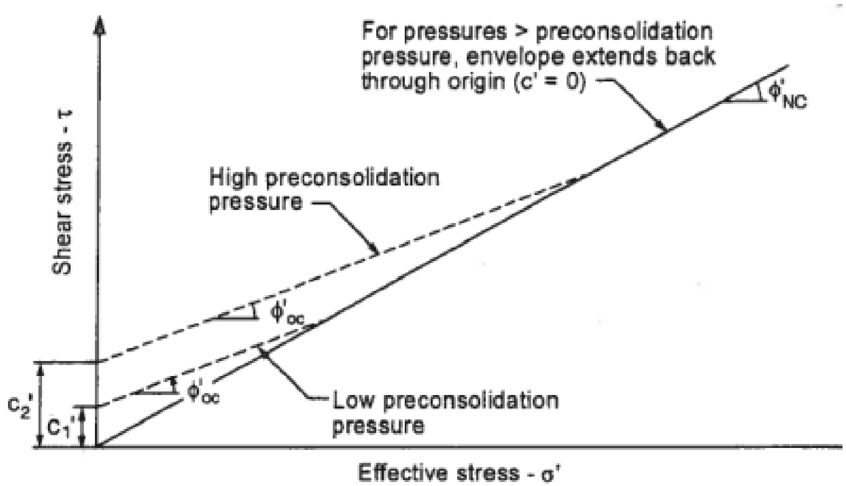


Figure 5-8: Drained Mohr-Coulomb Shear Strength Envelop for Clays (Duncan et al. 2014)

Drained Mohr-Coulomb shear strength envelopes for stiff-fissured clays are presented in Figure 5-9. Stiff-fissured clays are heavily overconsolidated clays that are typically stiff and contain fissures. These clays can exhibit peak, fully softened, and residual drained shear strengths. An undisturbed clay specimen tested in the field or laboratory will exhibit a peak drained shear strength around a shear displacement of 0.1 to 0.25 inch, or 3 to 6 millimeters (Duncan et al., 2014). As displacement continues beyond the peak, the shearing resistance of the specimen decreases to a residual value (at a displacement of about 10 inches, or 250 millimeters), as shown in Figure 5-9.

A slickensided surface along the failure plane is generally formed when the specimen reaches its residual shear strength. The clay will exhibit a fully softened drained shear strength if the same specimen is remolded and mixed with enough water to raise its water content to the liquid limit. The fully softened strength is lower than the undisturbed peak strength. As the remolded specimen is sheared and displacement continues beyond the fully softened strength, the shearing resistance of the specimen decreases to the same residual value as the undisturbed specimen. The undisturbed peak shear strength is generally not used to evaluate the stability of slopes composed of stiff-fissured clays. Shear strengths that can be mobilized in the field are generally less than in the laboratory since more softening and swelling occurs in stiff-fissured clays in the field over long periods of time. The fully softened shear strength is generally more appropriate to account for swelling and softening of the clay. However, if a failure has occurred and a slickensided failure surface has developed, only the residual shear strength can be mobilized to resist sliding and should be used in stability analyses.

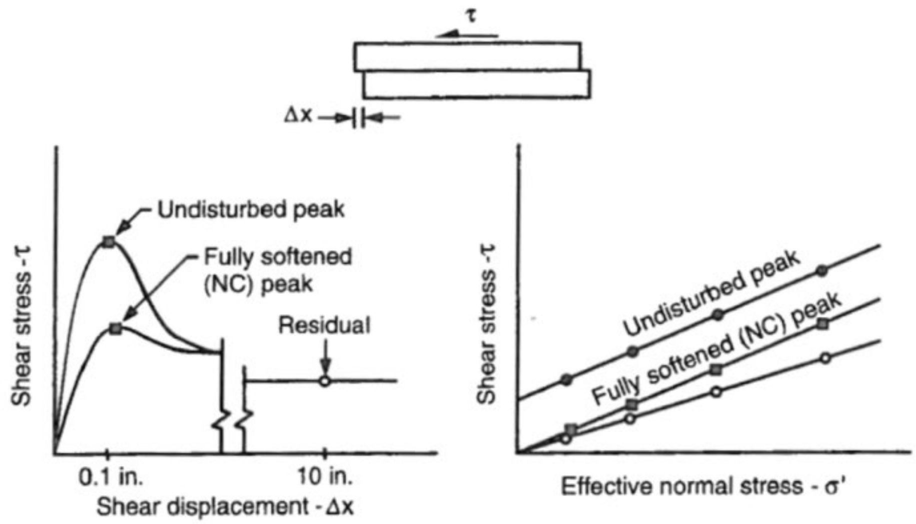


Figure 5-9: Drained Shear Strength of Stiff-Fissured Clays (Duncan et al. 2014)

Other factors affecting clay shear strengths include anisotropy and strain rate. The undrained shear strength of clays varies with the orientation of the principal stresses at failure and with the orientation of the failure plane. Figure 5-10(a) illustrates principal stress orientations at failure around a shear surface (i.e., slip surface). The undrained shear strength varies along the shear surface. Figure 5-10(b) shows variations in undrained shear strengths with orientation of the applied stress from unconsolidated-undrained (UU) triaxial tests on two normally consolidated clays and two heavily overconsolidated clay shales. Furthermore, undrained shear strengths evaluated through laboratory testing can sometimes be overestimated due to higher strain rates used to fail the specimen compared to those in the field.

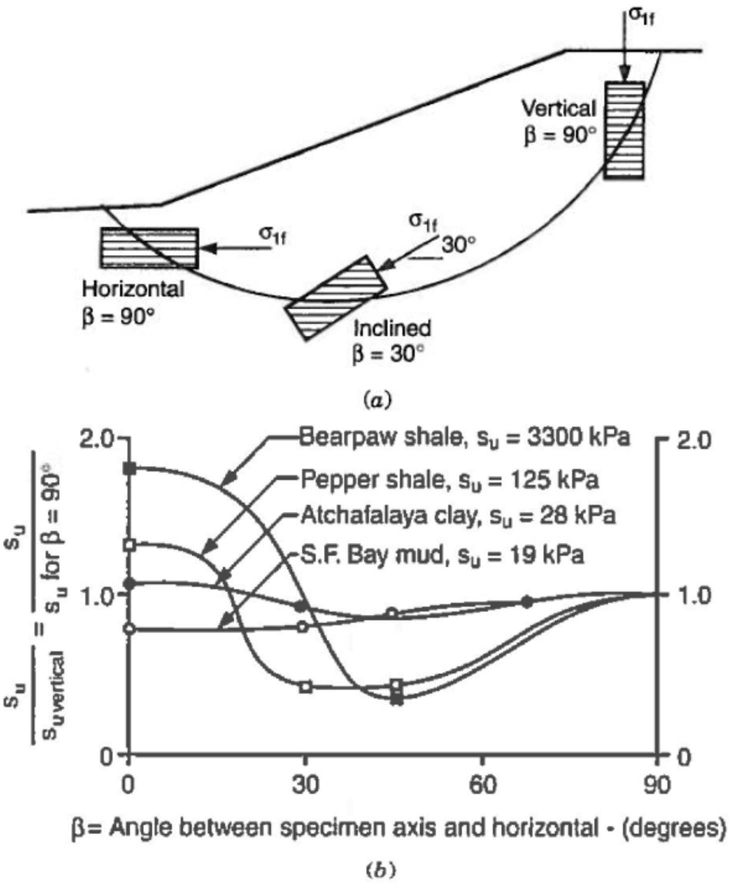


Figure 5-10: Anisotropy Effects for Clays – (a) Stress Orientations at Failure and (b) Undrained Shear Strength Anisotropy of Clays and Shales – UU Triaxial Tests (Duncan et al. 2014)

Laboratory tests used to measure the undrained shear strength (ϕ, c, or S_u) of clays include the unconfined compression (UC) test (S_u), UU triaxial shear test (ϕ, c, or S_u), CU'/CU triaxial shear test with or without pore pressure measurements (ϕ, c, or S_u), and the direct simple shear (DSS) test (S_u). Sample disturbance can reduce the undrained shear strength measured in laboratory tests. This effect may be reduced if the sample is consolidated to the same confining pressure it was consolidated to in the field. The SHANSEP (Stress History and Normalized Soil Engineering Properties) method can also be used to compensate for sample disturbance and is a common approach used to estimate the undrained shear strength of clays. As described by Ladd and Foot (1974) and Ladd et al. (1977), the method involves consolidating clay samples to effective stresses that are greater than the in-situ stresses and interpreting measured strengths in terms of an undrained shear strength ratio (S_u/σ'_v). Figure 5-11 shows the variation of S_u/σ'_v with OCR for six clays.

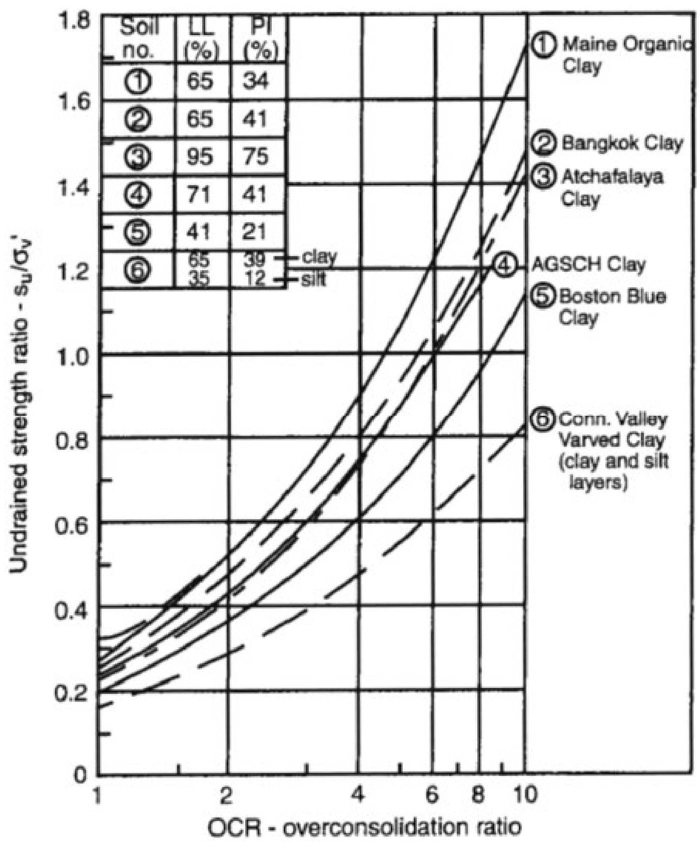


Figure 5-11: Variation of S_u/σ'_v with OCR for Clays, measured in Anisotropically Consolidated DSS Tests (Duncan et al. 2014)

The equation used to evaluate S_u/σ'_v for clays that normalize under the SHANSEP procedure is expressed by:

$$\frac{S_u}{\sigma'_v} = S(OCR)^m$$

Where: σ'_v = Effective Vertical Consolidation Pressure before Load Is Applied
S = Undrained Strength Ratio for Clay in Normally Consolidated Condition
OCR = Overconsolidation Ratio
m = Empirical Exponent

In the absence of site-specific data for the clay, Jamiolkowski et al. (1985) proposed that a value of 0.23 could be used for S and 0.8 for m. Shear strength ratios developed using typical CU' triaxial tests (where the samples are not consolidated to effective stresses that are greater than the in-situ stresses and backed off to known OCR values) will result in linear strength envelopes for clay samples that are normally consolidated at the time of shearing and curved envelops for clay samples that are overconsolidated at the time of shearing. Again, strength envelopes with intercepts are a mathematical convenience. It is important to note that undrained strength assignments using undrained shear strength ratios should be made using the steady-state effective stresses before the load is applied. This can be accomplished using a staged analysis approach, where the total stress analysis is completed for the second stage applied loads and the undrained strength of the material is calculated based on stresses in the first stage (before loading). As discussed in Section 5.4.1.2, the analysis procedure for the rapid drawdown loading condition by Duncan et al. (1990) uses this approach to assign undrained shear strengths.

A field test used for direct measurement of S_u for clays is the vane shear test, which has been successfully used for measuring the undrained shear strength of soft to medium-stiff clays. Limitations of the vane shear test are that it can be affected by sand lenses and seams, and the raw undrained shear strength measured from the test requires an empirical correction factor that varies with plasticity index and accounts for anisotropy and strain rate effects. The data that provide the basis for the correction factor are widely scattered, and therefore, vane strengths should not be viewed as precise. The pocket penetrometer test and Torvane test can be used to obtain quick, approximate measurements of undrained shear strength in the field or laboratory. However, the pocket penetrometer and Torvane tests are relatively crude and should be considered as only rough indications of shear strength.

The laboratory test most commonly used to measure the drained shear strength (ϕ', c') of clays is the CU' triaxial shear test. The CU' triaxial test is more practical than the CD triaxial test because the strain rates required for a CD test are typically extremely slow, requiring an impractically long test time. In addition, the CU' test can be used to obtain both undrained (total stress) and drained (effective stress) shear strength parameters. However, the CD triaxial test and DS test can also be used if the long test times can be accommodated. For stiff-fissured clays, laboratory tests are performed on remolded specimens to evaluate fully softened and/or residual drained shear strengths. The DS test is commonly used to measure the fully softened shear strength of stiff-fissured clays. Torsional ring shear tests are most suitable for estimating the residual shear strength of stiff-fissured clays because the test can measure shear stresses over any magnitude of displacement through continuous rotation. For more information on laboratory and field shear strength tests, refer to additional references in Section 7 (AECOM 2014).

Laboratory tests provide the best strength data for clays, and reasonably undisturbed samples of these soils can typically be obtained. However, various empirical correlations developed to estimate strengths for clays may be sufficient in some cases. It is recommended that these methods and correlations be used with caution because the behavior, and hence strength characterization, for clays is typically complex and may not be appropriately captured by the correlations. A few of the more common empirical shear strength correlations are presented in Duncan et al. (2014). It may also be useful to use correlations as a check to validate the results of laboratory tests.

Silts – Silts have an interesting soil particle composition, as they can behave similar to either fine sands or clays. When the term "fine-grained" is used, it almost always includes silts in this category. But silts themselves can be divided into two general categories: non-plastic and plastic. Non-plastic silts behave more like fine sands, while plastic silts behave more like clays. Evaluating whether a unit of silt is non-plastic or plastic can be achieved using the Atterberg Limit laboratory test.

Since silts can have a wide range of permeabilities, it can be difficult to predict if these soils will display drained or undrained behavior under various loading conditions. It is common to characterize silts using both drained and undrained shear strengths, similar to clays. For drained conditions, the shear strength of silts can be characterized by an effective stress friction angle (ϕ') with an assumed effective stress cohesion (c') equal to zero. For undrained conditions, the shear strength of silts can be expressed using total stress strength parameters (ϕ, c) or in terms of undrained strength (S_u). Similar to clays, there are different forms of characterization that can be used for S_u, for example, constant S_u, S_u as a function of effective confining stress, and S_u as a function of depth.

Laboratory tests used to measure values of ϕ' for silts include the CD and CU' triaxial shear tests and the DS test. Laboratory tests used to measure the undrained shear strength (ϕ, c, or S_u) of silts include the UC test (S_u), UU and CU'/CU triaxial shear tests (ϕ, c, or S_u), and the DSS test (S_u). Similar to coarse-grained soils, it can be difficult to obtain quality undisturbed samples of silts in the field, particularly non-plastic or very low plasticity silts.

Strength behaviors of silts have not been as widely studied as those of sands or clays. As a result of this general lack of research and compilation of data, very few empirical correlations exist for predicting shear strength values for silts. Empirical shear strength correlations that are available for silts are often regionally specific and developed with relatively limited data sets. It would be prudent to incorporate a level of conservatism when using these correlations for silts.

Similar to sands and clays, SPT, CPT, and shear wave field tests can be used for empirical shear strength correlations of silts. Empirical correlations using results of field tests for sands can generally be applied to estimate shear strengths of non-plastic silts. Shear strengths of plastic silts can generally be estimated from empirical correlations using results of field tests for clays.

5.4.2.3 Phreatic Surface

In addition to material properties, slope stability modeling requires inputting a phreatic surface through the embankment dam, which involves an evaluation of pore water pressures through the embankment and foundation. The evaluated phreatic surface is typically associated with the steady-state loading condition. In cases where the pore pressures vary significantly between the embankment and foundation, it may be acceptable to apply two separate piezometric surfaces, one assigned to the embankment (phreatic surface) and one assigned to the foundation (potentiometric surface). Because instability in embankment dams is often preceded by seepage problems, it is essential to understand and capture the seepage conditions occurring through the embankment and foundation in the stability model. Pore water pressures and the resulting phreatic surface through the embankment can be estimated from piezometer and/or monitoring well data or seepage analyses.

When piezometer and/or monitoring well data are used to estimate pore water pressures and the resulting phreatic surface, it is beneficial to plot the available piezometric measurements with measurements of reservoir level, as shown in Figure 5-12, to evaluate the fluctuation in piezometric water levels with reservoir filling and drawdown cycles. The phreatic surface is typically estimated using the maximum water levels recorded in the piezometers, which should generally correspond to the maximum normal reservoir level. It may also be helpful to plot the approximate phreatic surface (and potentiometric surface if present) on a cross section of the embankment before evaluating the stability model to guide critical thinking about how the embankment may perform.

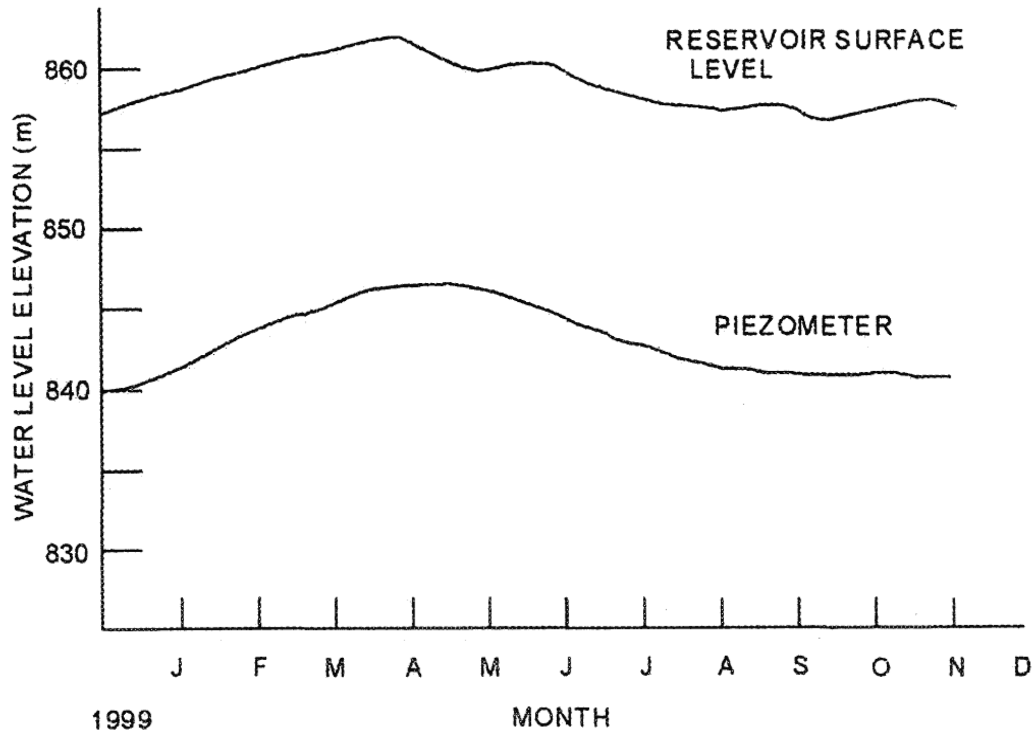


Figure 5-12: Typical Piezometer Data Plot

For irrigation reservoirs that experience fluctuations in water levels annually, the piezometric response to reservoir level must be carefully evaluated before assuming a steady-state phreatic surface. Typically, there is a lag between reservoir water level fluctuations and piezometer response. When modeling irrigation reservoirs, carefully look at lags in piezometric responses as the reservoir fills, reaches full pool, and starts to draw down. A 2-week lag is common. If the reservoir is not at full pool at least this long, assuming a steady-state phreatic surface may not be fully representative but is usually conservative.

When seepage analyses are used to estimate pore water pressures and the resulting phreatic surface, the results can be directly input into the stability model. Seepage analyses are discussed further in Section 3 of this guidance document. Seepage analyses for slope stability modeling are typically only performed when current, sufficient, and reliable piezometer data are not available to evaluate seepage response in existing embankment dams, or for new embankment dams in the design process or recently constructed dams for which steady-state conditions with sufficient piezometric measurements have not yet been developed.

For cases when piezometer data are not available or when a seepage analysis is too costly to perform, it may be appropriate to approximate the phreatic surface based on engineering judgment and experience with similar embankment configurations. For examples on how to draw typical steady state phreatic surfaces, see Section 3.2.1.2, and for additional reading, see Section 7 (Cedergren 1989; NRCS 1979; NRCS 1973). Sensitivity analyses can also be performed for a potential range of pore pressures to compensate for uncertainty.

5.4.2.4 Desktop Review

A desktop review of all available information for an embankment dam should be completed to determine whether there are sufficient data on the embankment geometry, internal zoning, foundation contact and stratigraphy, and material properties to perform the slope stability analysis. The desktop review can also identify potential data gaps. FEMA (2015b) presents the following checklist for guidance on conducting a desktop review:

Desktop Review Checklist

Design analyses and assumptions (original and any modification or remedial designs)	☐ Yes	☐ No	Photographs	☐ Yes	☐ No
Original geologic and geotechnical reports	☐ Yes	☐ No	As-built drawings	☐ Yes	☐ No
Any data related to seepage analysis, filter design, stability analyses	☐ Yes	☐ No	Operation and maintenance records	☐ Yes	☐ No
Construction plans and specifications	☐ Yes	☐ No	Instrumentation records	☐ Yes	☐ No
Construction reports, logs, and records	☐ Yes	☐ No	Past inspection reports	☐ Yes	☐ No
			Any correspondence from regulators relative to deficiencies or problems	☐ Yes	☐ No

If data gaps are identified, additional investigations or studies may be warranted. For information on performing geotechnical investigations in embankments, including typical drilling and sampling methods, refer to additional references in Section 7 (AECOM 2014a, 2014b, 2014c, 2015; FEMA 2015b).

5.4.3 Model Slip Surface Considerations

In a limit equilibrium slope stability model, the factor of safety is calculated for a trial slip surface and is assumed to be constant along the slip surface. A number of trial slip surfaces must be evaluated to identify the critical slip surface, which is the surface resulting in the lowest (or minimum) calculated factor of safety. Critical considerations for evaluating trial slip surfaces in a slope stability model include the slip surface configuration and location, as well as tension cracks. These considerations are discussed further below.

5.4.3.1 Slip Surface Configuration and Location

Evaluating the configuration (or shape) and location of the critical slip surface corresponding to the minimum factor of safety is one of the greatest challenges in slope stability modeling and requires a trial procedure. The slip surface configuration and location will depend on the embankment geometry and internal zoning, foundation stratigraphy, material characteristics (including shear strength and anisotropy), and pore water pressure conditions.

Two common slip surface configurations for slope stability modeling are circular and noncircular slip surfaces, as shown in Figure 5-13. Circular slip surfaces are defined by an arc of a circle that cuts through the slope and are typically applicable for evaluating relatively homogeneous or zoned embankments founded on thick soil deposits. Noncircular slip surfaces are defined by connected linear segments that cut through the slope and are typically more applicable for evaluating zoned embankments on layered foundations, particularly with weak seams. A wedge slip surface is a type of noncircular surface defined by three linear segments and may be appropriate for evaluating a relatively long, horizontal weak seam bounded by stronger materials. Due to modern computing capabilities, stability modeling using noncircular slip surfaces is very common and useful for complex geometries.

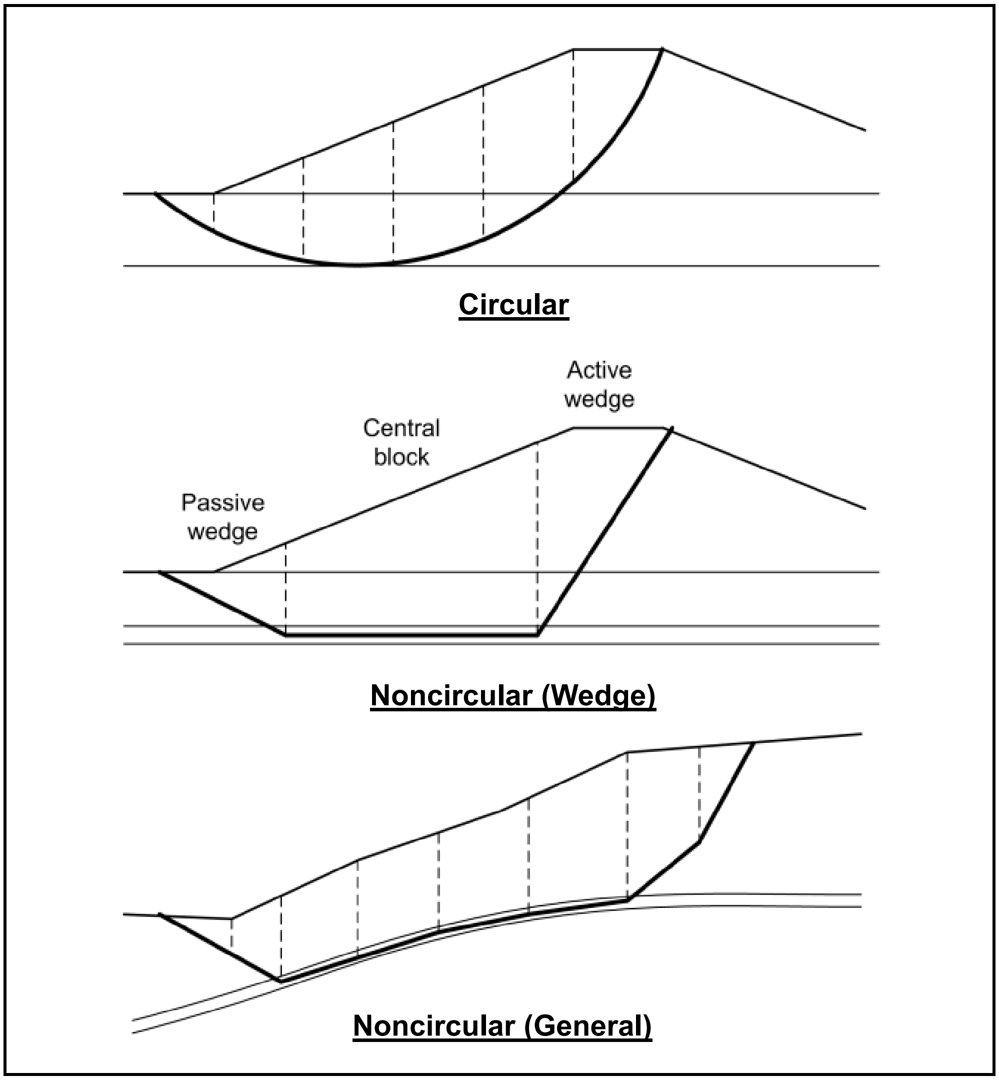


Figure 5-13: Common Slip Surface Configurations (USACE 2003)

The critical slip surface location is related to zones of relatively weaker materials and/or higher pore water pressures. Automatic search procedures used in computer programs can be performed for both circular and noncircular slip surfaces and aid in locating the critical slip surface corresponding to the minimum factor of safety. However, considerable judgment must be exercised to ensure that the most critical slip surface has been located. Details of the search procedures vary among computer programs and should be understood prior to the analysis. Most search procedures require an initial estimate of the starting location of the slip surface. For circular slip surfaces, the starting location is typically defined by the x and y coordinates of the center of the circle and the radius of the circle. For noncircular slip surfaces, the starting location is defined by the x and y coordinates for each point on the slip surface.

Provided below are tips for consideration.

- Tip 1 – Conduct automatic searches for trial slip surfaces at several starting locations to fully explore the soil profile and detect multiple local minimums. Multiple trial slip surfaces can be evaluated to verify that the global minimum factor of safety has been found.

Consider evaluating slip surfaces that pass through the embankment only and also those that pass through both the embankment and foundation.

- Tip 2 – Estimate the starting location of noncircular slip surfaces by first evaluating circular slip surfaces or by identifying locations of weak materials or seams.
- Tip 3 – Evaluate a range of local, intermediate, and global slip surfaces, as shown in Figure 5-14. The factor of safety can vary between local slip surfaces that pass through a limited portion of the embankment slope or toe area and global slip surfaces that pass through the embankment crest and encompass the full slope and toe area.
- Tip 4 – Examine slip surfaces and factors of safety that are not minimums. The slip surface with the minimum factor of safety is not always the one of greatest interest. Often, shallow infinite slip surfaces have a lower factor of safety than deeper slip surfaces but are not considered to have a global stability impact or impact the safety of dams. Failure along a shallow surface may consist of material raveling downslope and presenting merely a maintenance issue, whereas failure along a deeper surface can have more severe consequences.

MATERIAL PROPERTIES

Material Description	Total Unit Weight (pcf)	Effective Strength Parameters	
		ϕ' (deg)	c' (psf)
Unsaturated Embankment	125	33	50
Saturated Embankment	130	33	0
Toe Drain	120	35	0
Alluvium	125	32	0
Bedrock	150	45	5000

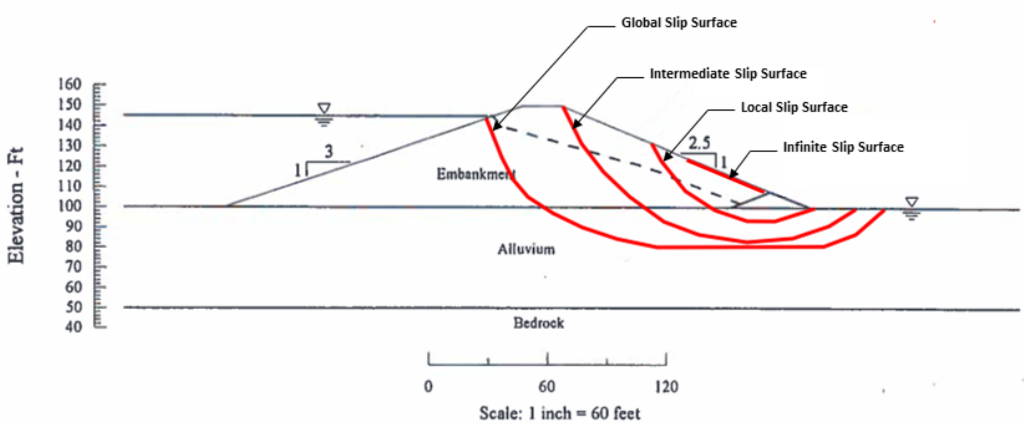


Figure 5-14: Examples of Local, Intermediate, and Global Slip Surfaces for Slope Stability Modeling

5.4.3.2 Tension Cracks

Steep slip surface angles at the entrance and exit of the evaluated slope can cause tensile forces to develop at the interfaces between slices and on the bottom of slices at the entrance and exit areas. These tensile forces can sometimes cause model convergence issues or numerical problems in calculating the factor of safety. As discussed in Section 5.1.1, soils do not generally exhibit significant tensile shear strength and thus cannot withstand tension. Therefore, the presence of tension in slope stability models is considered unrealistic. Tension can imply tensile strength that does not exist and will result in a calculated factor of safety that is too high. The adverse effects of tension can be eliminated from the calculations of stability modeling by introducing a tension crack. A tension crack terminates the slip surface at the edge of a slice at an appropriate depth below the ground surface, as shown in Figure 5-15. The depth of a tension crack can be estimated from simple equations derived from earth pressure theory. In most cases, only an approximation of the depth of tension crack is needed. Typically, a tension crack is introduced into the slope stability model, and the depth is varied until convergence is satisfied. For more information on tension cracks and calculating the depth of a tension crack, refer to additional references in Section 7 (Duncan et al. 2014).

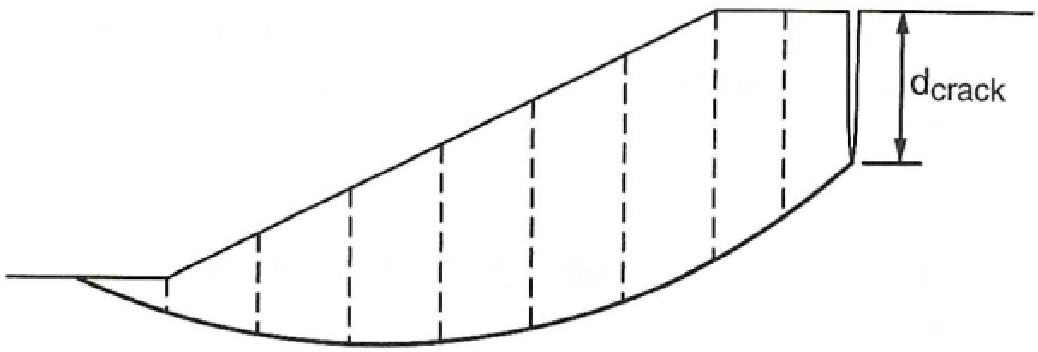


Figure 5-15: Slope and Slip Surface with Tension Crack (Duncan et al. 2014)

5.4.4 Model Sensitivity Considerations

Slope stability modeling is influenced by several factors, including variations in unit weight, shear strength, and phreatic surface conditions. Modelers should understand which factors have greater impacts on the results of slope stability analyses so that more effort can be spent estimating the most influential factors. The sensitivities of variations in unit weight, shear strength, and phreatic surface conditions were evaluated by France and Winckler (2010) and are discussed below.

5.4.4.1 Unit Weight and Shear Strength

The effects of variations in unit weight and shear strength on the results of slope stability analyses were evaluated using two example stability models presented in Figure 5-16 and Figure 5-17 for a base case condition and sensitivity case. The base case condition used the materials properties (i.e., unit weights and drained effective stress shear strengths) shown in Figure 5-16 and Figure 5-17 and a phreatic surface estimated using an anisotropic permeability ratio (i.e., ratio of horizontal to vertical permeability) of $4(k_h):1(k_v)$. The slope stability results for the two examples under the base case condition are presented in Figure 5-18 and Figure 5-19, which indicate calculated minimum factors of safety for a global (i.e., deep-seated) slip surface of 1.91 and 1.66, respectively.

		EFFECTIVE STRENGTH PARAMETERS		
NO.	MATERIAL DESCRIPTION	UNIT WEIGHT (pcf)	φ' (deg.)	C' (psf)
1	UNSATURATED EMBANKMENT	125	33	50
2	SATURATED EMBANKMENT	130	33	0
3	TOE DRAIN	120	35	0
4	BEDROCK	150	45	5,000

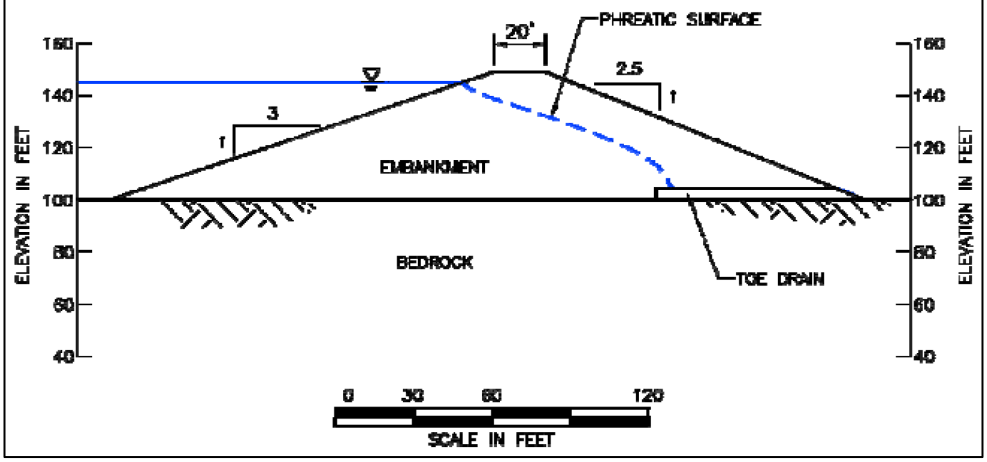


Figure 5-16: Example 1 – Slope Stability Model for Homogeneous Embankment with Toe Drain (France and Winckler 2010)

	EFFECTIVE STRENGTH PARAMETERS		
MATERIAL DESCRIPTION	UNIT WEIGHT (pcf)	φ' (deg.)	C' (psf)
SHELL	130	33	0
FILTER	125	33	0
CORE	110	35	0
BEDROCK	150	45	5,000

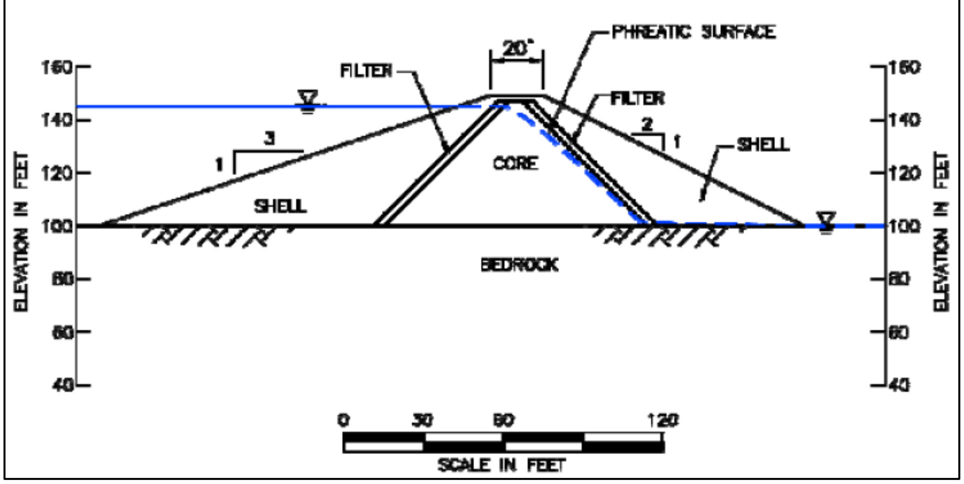


Figure 5-17: Example 2 – Slope Stability Model for Zoned Embankment with Chimney and Blanket Drains (France and Winckler 2010)

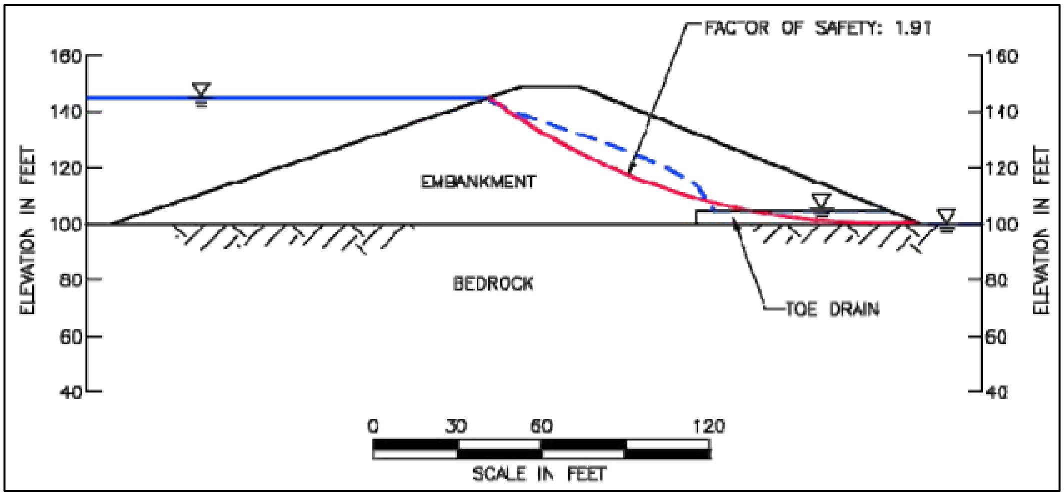


Figure 5-18: Slope Stability Model Results for Example 1, Base Case Condition (France and Winckler 2010)

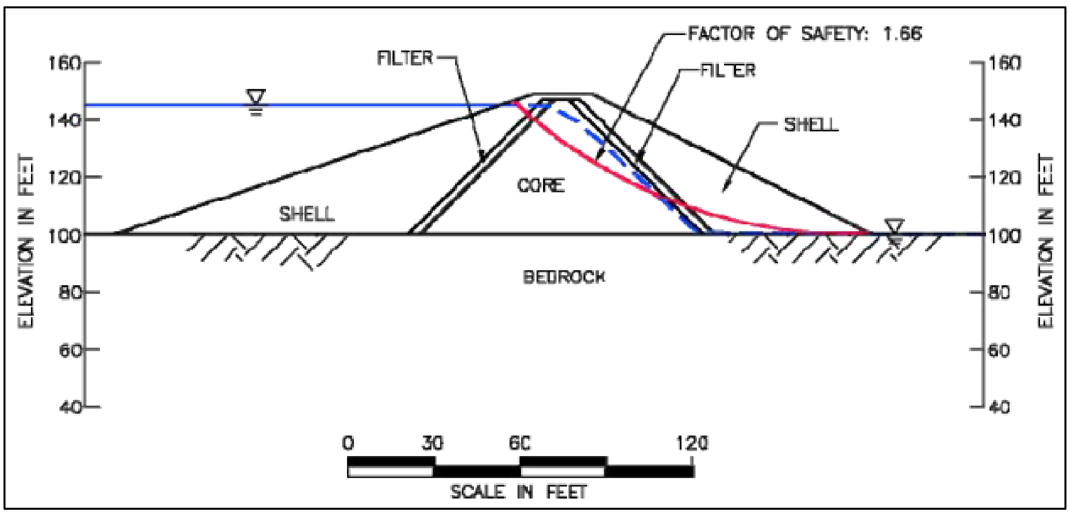


Figure 5-19: Slope Stability Model Results for Example 2, Base Case Condition (France and Winckler 2010)

The sensitivity case varied the unit weights and shear strengths of all the materials independently by ±5%, ±10%, and ±20%. The variations in strengths were applied to both the tangent of the effective stress friction angle (tanϕ’) and the effective stress cohesion (c’). The slope stability model results for the two examples under the sensitivity case incorporating independent variations in unit weight and shear strength are summarized in Table 5-6 and Table 5-7, respectively.

Table 5-6: Slope Stability Model Results for Sensitivity Case – Variations in Unit Weight (France and Winckler 2010)

Variation in Unit Weight	Minimum Calculated Factor of Safety	
	Example 1	Example 2
-20%	1.84	1.62
-10%	1.88	1.64
-5%	1.89	1.65
Base Case	1.91	1.66
+5%	1.92	1.66
+10%	1.93	1.67
+20%	1.95	1.67

Table 5-7: Slope Stability Model Results for Sensitivity Case – Variations in Strength (France and Winckler 2010)

Variation in Shear Strength	Minimum Calculated Factor of Safety	
	Example 1	Example 2
-20%	1.53	1.33
-10%	1.72	1.49
-5%	1.81	1.58
Base Case	1.91	1.66
+5%	2.00	1.74
+10%	2.10	1.82
+20%	2.29	1.99

Based on the results, the following observations were noted:

- For variations in shear strength, changes in the minimum factors of safety are approximately proportional to changes in strength.
- For variations in unit weight, changes in the minimum factors of safety are much less than proportional to changes in unit weight.

Therefore, the results of slope stability analyses are significantly more sensitive to variations in shear strength than unit weight, and more time should be spent estimating the strengths of the materials. It may also be important to approach the selection of shear strengths for stability analyses in a conservative manner. Alternatively, sensitivity analyses can be performed for a potential range of shear strengths to compensate for uncertainty.

5.4.4.2 Phreatic Surface

Similar to the unit weight and shear strength, the effect of variations in the phreatic surface (i.e., internal water levels) on the results of slope stability analyses were evaluated using the example stability model presented in Figure 5-16 above. Phreatic surfaces were estimated using increasing anisotropy ratios (i.e., ratios of horizontal to vertical permeability) of 1, 4, and 9. The phreatic surfaces corresponding to these three anisotropy ratios are illustrated in Figure 5-20.

The slope stability model results for the example under variations in the phreatic surface are summarized in Table 5-8. The results indicate that the factor of safety significantly decreases under higher phreatic surfaces associated with increasing anisotropy ratios. Therefore, the results of slope stability analyses are sensitive to variations in the phreatic surface, and time should be spent estimating the phreatic surface from available piezometer data or seepage analyses. It may also be important to approach the estimation of the phreatic surface for stability analyses in a conservative manner. Alternatively, sensitivity analyses can be performed for a potential range of pore water pressures (i.e., phreatic surface conditions) to compensate for uncertainty and/or a piezometer threshold analysis can be performed, as discussed in Section 5.4.1.5.

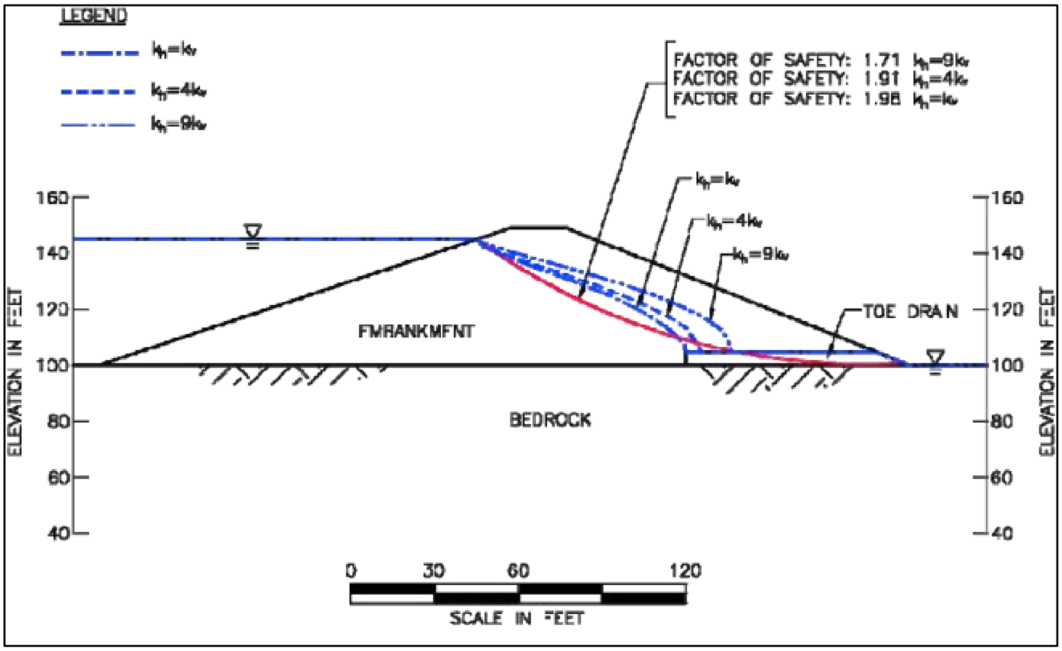


Figure 5-20: Phreatic Surfaces Corresponding to Increasing Anisotropy Ratios and Slope Stability Model Results for Variations in Phreatic Surface (France and Winckler 2010)

Table 5-8: Summary of Slope Stability Model Results for Variations in Phreatic Surface (France and Winckler 2010)

Anisotropy Ratio, kh/kv	Minimum Calculated Factor of Safety
1	1.98
4	1.91
9	1.71

5.4.5 Model Convergence Considerations

For limit equilibrium slope stability analyses, model convergence issues occur when the model cannot obtain moment or force equilibrium. Convergence issues are commonly due to high shear strength contrasts between zones, sharp corners in the material or slip surface geometry, and point loads. Starting simple is a good way to overcome convergence issues. There is no one solution that fits all situations of non-convergence. However, the following tips may be helpful:

- Tip 1 – Simplify the model if it makes sense to do so.
- Tip 2 – Reduce high shear strength contrasts.
- Tip 3 – Remove sharp corners.
- Tip 4 – Distribute point loads.
- Tip 5 – Try different slip surface shapes and search methods.

If convergence issues do occur, an error message will typically display, or the model will not run. However, on very rare occasions, the authors have found that some models will run with convergence issues. Red flag warnings include an unreasonably low factor of safety and lack of slip surfaces propagating through the weakest material.

5.5 Interpreting, Verifying, and Reporting Results

Although not as complex as seepage modeling, slope stability modeling is still complex and requires practice and experience. Models will often run without error and produce professional-looking results but can be invalid. Successful runs should not be confused with accurate results. Interpreting and verifying the results are not always intuitive to the novice user. Vetting errors and understanding the sensitivity of a model to the potential range of each input parameter should be a priority before using the results. This often requires knowledge gained through personal trial and error. Consider seeking guidance from experienced engineers and modelers. If not available in-house, program vendors and developers often offer limited technical support for vetting specific problems.

Specific situations may warrant hiring an expert to perform the slope stability modeling. It is easier to create models for the steady-state loading condition than other conditions, but most embankments require an evaluation of all loading conditions. Shear strengths need to be characterized differently for the various loading conditions. Shear strength is the single most influential parameter in a slope stability analysis, yet also the most complex to characterize, especially for undrained strengths. Some specific situations that may warrant hiring an expert are as follows:

- Transient conditions,
- Complex model geometry with high shear strength contrast,
- Material strength variability within the core or foundation,
- High consequences (i.e., model is used to direct a significant design decision), or
- Modeling reinforcements or point loads.

There may also be merit in getting a secondary review from an expert, as some regulatory agencies may not have the modeling experience to catch problems.

The following sections provide tips, tools, and guidance on how to interpret and verify slope stability analysis results, as well as report the results.

5.5.1 Interpreting and Verifying Results

After the slope stability model is run and results are obtained, it is important to examine the calculated minimum factor of safety and corresponding critical slip surface to understand and verify the model results. Tips for checking the validity of the model results include the following:

- Research the computer program being used for stability modeling to understand the theory, methodology, assumptions, and weaknesses inherent to the program. Having adequate knowledge of the program will allow for better interpretation of the results.
- Plot multiple trial slip surfaces together on the analyzed embankment cross section to verify the critical slip surface is reasonable. The configuration and location of the critical slip surface is dependent on zones of weaker materials. For a zoned embankment with a core material that is weaker than the shell material, the critical slip surface is likely to pass through the core. For an embankment on a layered foundation with a weak seam, the critical slip surface is likely to be a noncircular surface that passes through the weak seam.
- Compare the results to past performance of the dam. If the calculated minimum factor of safety for the steady-state loading condition is less than 1.0 but the embankment has historically performed well without stability issues, the results may be invalid and may warrant further evaluation of model inputs, such as material properties (e.g., shear strengths) and/or the phreatic surface.
- Use a second computer program, slope stability charts, and/or detailed hand calculations to verify similar results.
- Perform sensitivity analyses to understand the sensitivity of the model to a potential range of each input parameter. Sensitivity analyses are also useful for evaluating uncertainties in shear strengths, pore water pressures, etc.

An example slope stability model using the UTEXAS4 computer program is presented in Figure 5-21 and is used here for providing additional guidance in interpreting results. The example model is for a homogeneous embankment dam founded on relatively strong sandstone bedrock with two weak interbedded horizontal clay seams. The phreatic surface through the embankment intersects with a toe drain at the downstream portion of the embankment.

	Effective Strength Parameter		
Material Description	Unit Weight (pcf)	φ' (deg.)	C' (psf)
Unsaturated Embankment	125	33	50
Saturated Embankment	130	33	0
Toe Drain	120	35	0
Bedrock	150	45	5,000
Clay Seam 1	150	13	0
Clay Seam 2	150	8	0

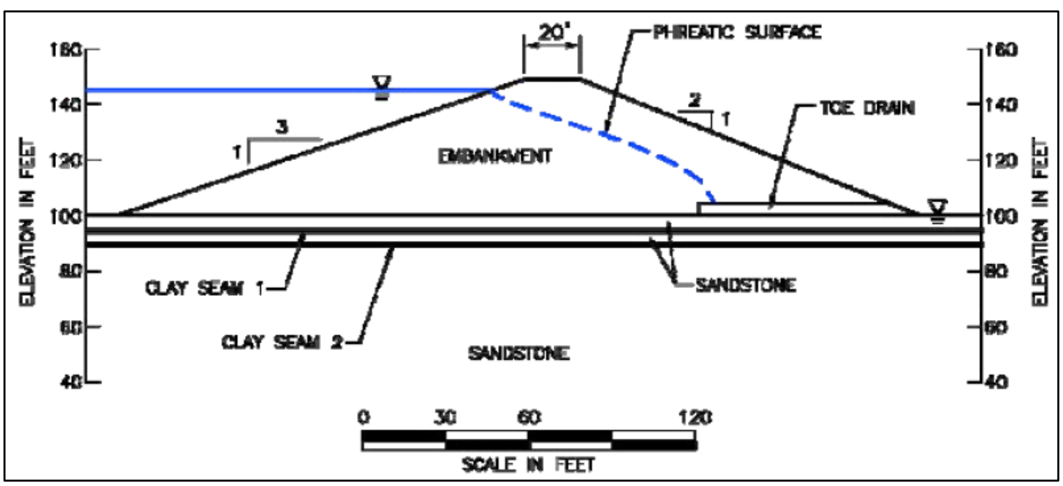


Figure 5-21: Example Slope Stability Model Using UTEXAS4 (France and Winckler 2010)

The validation and interpretation of the slope stability model results (shown in Figure 5-22) are summarized as follows:

- Figure 5-22 illustrates the resulting factors of safety for trial slip surfaces using three different search methods: (1) circular search, (2) noncircular search through the upper weak clay seam, and (3) noncircular search through the lower weak clay seam.
- The circular search resulted in an intermediate slip surface confined to the embankment, with a calculated factor of safety of 1.78. The noncircular searches resulted in global slip surfaces passing through each of the weak clay seams, with significantly lower calculated factors of safety of 1.21 through the upper weak seam and 1.11 through the lower weak seam. Thus, the critical slip surface corresponding to the minimum calculated factor of safety is the global noncircular slip surface that passes through the lower weak seam. This should be expected and is reasonable since the lower clay seam is weaker than the upper clay seam, as well as the other embankment and foundation materials. Furthermore, the noncircular slip surfaces have more adverse pore water pressures compared to the circular slip surface. The noncircular surfaces are completely below the phreatic surface, while the circular surface is mostly above the phreatic surface.

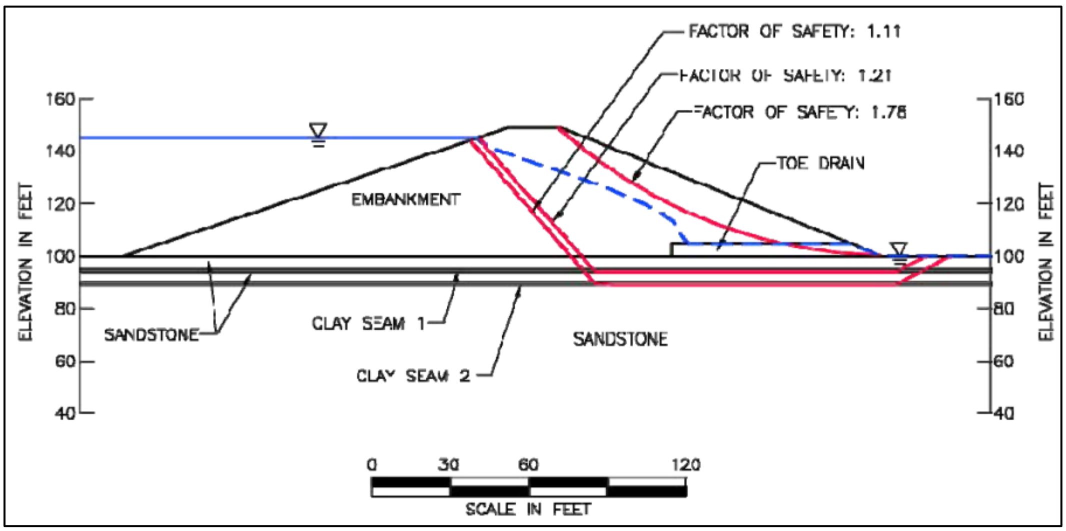


Figure 5-22: Slope Stability Model Results Showing Factors of Safety for Trial Slip Surfaces (France and Winckler 2010)

The slope stability model input and output (results) files should also be critically reviewed to verify that the following conditions are true for the critical slip surface (with spot checks for surfaces other than the critical surface):

- There are no outstanding errors or warnings.
- Model geometry is correct.
- Materials are assigned the correct material properties.
- Pore pressure is calculated correctly.
- Load conditions are being executed correctly.
- Shear stress for each slice is greater than or equal to zero, and no shear stress is unreasonably high (relative to the other slices). Shear stress should never be negative.

- The summation of horizontal forces, vertical forces, and moments are practically zero (i.e., summation will never be exactly zero).
- The slice calculations are using the correct material to estimate shear strength.

5.5.2 Reporting Results

Upon completion of a slope stability analysis, the analysis should be adequately documented in a report for submittal to the dam owner and/or regulator. The report should include sufficient detail for the reader to understand the purpose of the analysis, how the analysis was performed, the results of the analysis, and recommendations. Slope stability analysis reports should include a discussion on the following topics:

- Purpose – Clearly define the objective of the slope stability analysis. Refer to Section 5.3.
- Methodology – Describe the stability analysis method and approach (e.g., loading conditions, factor of safety criteria, 2D or 3D, computer program used). Refer to Section 5.4.1.
- Model Geometry – Summarize the embankment geometry and internal zoning and the foundation stratigraphy modeled in the stability analysis. Refer to Section 5.4.2.1.
- Material Properties – Summarize the stability material properties (e.g., unit weight and shear strength) assigned to the embankment and foundation materials and explain how they were evaluated. Refer to Section 5.4.2.2.
- Pore Water Pressures – Summarize the pore water pressure conditions used in the analysis and describe the source of the data (e.g. phreatic surface from piezometer data, pore pressures imported from seepage analysis). Refer to Section 5.4.2.3.
- Results – Provide summary tables and figures of the results and summarize the interpretation of the results. This may also include a discussion on the uncertainties of the evaluation and the results of sensitivity analyses to evaluate the uncertainties. Refer to Section 5.5.1.
- Recommendations – If applicable, provide recommendations for action (e.g., increased monitoring, reservoir storage restrictions, remediation) or additional investigations based on the results. The results may also be used for alternative design recommendations.

6. Conclusion

Seepage and slope stability modeling are often proposed as part of an embankment dam evaluation. These analyses can be valuable to a project when the objective is clearly defined, alternatives are considered, models are planned and executed properly, and results are interpreted and reported correctly.

This guidance document describes the standard of practice for seepage and slope stability analyses of embankment dams and provides tips, tools, and guidance on planning, modeling considerations, and interpreting, verifying, and reporting the results.

Basic seepage and slope stability concepts are summarized throughout the document to give the reader an understanding of how best to approach performing efficient, effective, and reliable seepage and stability models. References to additional publications that elaborate on these concepts are included in Section 7.

7. References and Additional Reading

AECOM. “Embankment Dam Slope Stability 101,” *Western Dam Engineering Technical Note*, Volume 1, Issue 3, November 2013 (sponsored by Colorado Division of Water Resources, Montana Department of Natural Resources, and Wyoming State Engineer’s Office). http://dnrc.mt.gov/divisions/water/operations/docs/dam-safety/technical-references/western_dam_engineering_technote-vol1issue3.pdf

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