RECLAMATION
Managing Water in the West

Design Standards No. 13

Embankment Dams

Chapter 8: Seepage
Phase 4 (Final)

U.S. Department of Interior
Bureau of Reclamation

January 2014

Mission Statements

The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Design Standards Signature Sheet

Design Standards No. 13

Embankment Dams

DS-13(8)-4.1: Phase 4 (Final)
January 2014

Chapter 8: Seepage

Revision Number DS-13(8)-4.1

Summary of revisions:

- A Foreword section was added before the Chapter Signature Sheet.
- In Section 8.2.2.2, "High Exit Gradients in a Cohesionless Soil," on page 8-7, in the table appearing on that page, the "Recommended Safety Values" in column 2 were changed.
- In Section 8.3.2.4, "Effect of Degree of Saturation on Permeability," on page 8-30, in the next to last paragraph on the page, the incorrect word "Note" was deleted.
- On the cover, title page, and footers, the date was changed from "October 2011" to "January 2014."
- Minor editorial corrections made throughout.

Prepared by:

1/24/2014

Bill Engemoen, P.E. — Date
Civil Engineer, Geotechnical Services Division

Peer Review:

Derek Wittwer — 1/24/2014

Derek T. Wittwer, P.E. — Date
Civil Engineer, Geotechnical Engineering Group 4

Submitted:

Karen Knight — 2/3/2014

Karen Knight, P.E. — Date
Chief, Geotechnical Services Division

Approved:

T. A. Luebke — 2/3/2014

Thomas A. Luebke, P.E. — Date
Director, Technical Service Center

Foreword

Purpose

The Bureau of Reclamation (Reclamation) design standards present technical requirements and processes to enable design professionals to prepare design documents and reports necessary to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public. Compliance with these design standards assists in the development and improvement of Reclamation facilities in a way that protects the public's health, safety, and welfare; recognizes needs of all stakeholders; and achieves lasting value and functionality necessary for Reclamation facilities. Responsible designers accomplish this goal through compliance with these design standards and all other applicable technical codes, as well as incorporation of the stakeholders' vision and values, that are then reflected in the constructed facilities.

Application of Design Standards

Reclamation design activities, whether performed by Reclamation or by a non-Reclamation entity, must be performed in accordance with established Reclamation design criteria and standards, and approved national design standards, if applicable. Exceptions to this requirement shall be in accordance with provisions of *Reclamation Manual Policy*, Performing Design and Construction Activities, FAC P03.

In addition to these design standards, designers shall integrate sound engineering judgment, applicable national codes and design standards, site-specific technical considerations, and project-specific considerations to ensure suitable designs are produced that protect the public's investment and safety. Designers shall use the most current edition of national codes and design standards consistent with Reclamation design standards. Reclamation design standards may include exceptions to requirements of national codes and design standards.

Proposed Revisions

Reclamation designers should inform the Technical Service Center (TSC), via Reclamation's Design Standards Website notification procedure, of any recommended updates or changes to Reclamation design standards to meet current and/or improved design practices.

Chapter Signature Sheet
Bureau of Reclamation
Technical Service Center

Design Standards No. 13

Embankment Dams

Chapter 8: Seepage

DS-13(8)-4.1[1] Phase 4 (Final)

Chapter 8 – Seepage is an existing chapter (last revised March 1987) within Design Standard 13 and was revised as follows:

- Expanded discussion of seepage analysis principles and procedures
- Expanded discussion of seepage mitigation measures
- Revised discussion of seepage-related failure modes
- Provided discussion for seepage monitoring
- Provided discussion of key data for seepage evaluations
- Added discussion of SEEP/W analyses and example problems
- Provided illustrative considerations and examples of exit gradients and uplift pressures at the downstream toe of the embankment

[1] DS-13(8)-4 refers to Design Standards No. 13, chapter 8, revision 4, minor intermediate revision 1.

Prepared by:

Bill Engemoen, P.E. — 10/27/2011 — Date
Civil Engineer, Geotechnical Services Division

Peer Review:

Derek T. Wittwer, P.E. — 10/28/11 — Date
Civil Engineer, Geotechnical Engineering Group 1

Security Review:

Larry K. Nuss, P.E. — 10/28/2011 — Date
Structural Engineer, Structural Analysis Group

Recommended for Technical Approval:

Thomas McDaniel — 10/27/2011 — Date
Geotechnical Engineering Group 2

Submitted:

Karen Knight, P.E. — 12/03/2012 — Date
Chief, Geotechnical Services Division

Approved:

Lowell Pimley, P.E. — 12/03/2012 — Date
ACTING FOR Director, Technical Service Center

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Symbols

I_c	Critical exit gradient (in vertical direction)
I_e	Predicted or measured exit gradient
γ_b	Buoyant unit weight of soil
γ_t	Total unit weight of soil
γ_w	Unit weight of water
G	Specific gravity of soil
e	Void ratio of soil
n	Porosity of soil
h	Hydraulic head or reservoir head that drives seepage (i.e., difference between reservoir and tailwater elevation)
k	Coefficient of permeability in saturated medium
k_{us}	Coefficient of permeability in unsaturated or partially saturated medium
k_H	Horizontal permeability
k_V	Vertical permeability
q	Seepage flow rate
i	Hydraulic gradient
A	Cross sectional area
V	Velocity of seepage flow
FS	Factor of safety

Chapter 8

Seepage

8.1 Introduction

8.1.1 Purpose

The purpose of this design standard chapter is to draw attention to critical failure modes associated with seepage and the important aspects of seepage control measures in the design of a new embankment dam or in the evaluation and/or modification of an existing embankment dam, and to present recommended practices for analysis of seepage issues and design of seepage control features.

8.1.2 Scope

This chapter places primary emphasis on seepage analysis as a means to: (a) understand and evaluate basic seepage-related issues that may impact a dam or its foundation; (b) predict seepage-related performance of an existing or new (or newly modified) embankment and its foundation; (c) assess the effectiveness of various seepage control features; (d) provide quantitative estimates, as well as general insights, for design of the selected seepage control features; and (e) provide the basic understanding needed for developing a prudent seepage monitoring program.

Note: Predicting seepage behavior is difficult due to the many variables involved in evaluating and modeling permeability and the often complex nature of foundation conditions. Seepage analyses are generally viewed as capable of predicting the general order of magnitude of results and *approximating* seepage behavior. As such, experience and judgment are essential in all phases of the evaluation of seepage behavior.

8.1.3 Deviations from Standard

Design and analysis of embankment dams within the Bureau of Reclamation (Reclamation) should adhere to concepts and methodologies presented in this design standard. Rationale for deviation from the standard should be presented in technical documentation for the dam and should be approved by appropriate line supervisors and managers.

8.1.4 Revisions of Standard

This standard will be revised periodically as its use and the state of practice suggests. Comments and/or suggested revisions should be sent to the Bureau of Reclamation, Technical Service Center, Attn: 86-68300, Denver, CO 80225.

8.1.5 Applicability

The guidance and procedures in this chapter are applicable to the analysis of various seepage issues related to embankment dams and their foundations. Guidance and discussion are provided on the types of failure modes that may result from seepage, data required to evaluate seepage problems, principles and procedures for analyzing seepage problems, various seepage mitigation measures, and important considerations for seepage monitoring. Discussion of specific details and criteria for the design of features to mitigate potential seepage concerns is beyond the scope of this chapter. However, references to other appropriate design standard chapters and/or examples of Reclamation facilities with specific design features are provided.

8.1.6 Definition of Terms

Following are the definition of terms used in this chapter.

8.1.6.1 Seepage

Seepage is considered to be all movement of water from the reservoir through the embankment, abutments, and foundation. This includes porous media (intergranular) flow, flow in fractures, and concentrated flow through “defects” such as cracks, loose lifts, etc.

8.1.6.2 Pore Pressure

Pore pressure is defined as water pressure in the voids of the embankment or foundation material and includes both the positive and the negative pore water pressure.

8.1.6.3 Phreatic Surface

The phreatic surface is the theoretical surface in the embankment and/or foundation along which the pressure head is zero.

8.1.6.4 Hydraulic Gradient

The hydraulic gradient at a point on the flow path is a vector with three components, i.e., $\partial h/\partial x$, $\partial h/\partial y$, $\partial h,\partial z$, where h is total hydraulic head (hydraulic head is pressure head plus elevation head). Frequently, gradient can be approximated by the total head difference between two points, divided by the distance between them; this gives the average gradient over that length. The actual gradient can vary significantly over short distances with changes in soil permeability or flow converging into a toe drain or well. In practical applications, use of the computed gradient should be carefully evaluated.

8.1.6.5 Permeability

Permeability (abbreviated as k) is defined as the *in situ* average rate at which a porous medium such as soil or rock can transmit water under unit hydraulic gradient and laminar flow conditions, in a given direction. Primary permeability refers to flow through the pore space (voids) of a soil or rock, while secondary permeability refers to flow through finite openings in the medium such as fractures or solution channels. Permeability can vary depending on the degree of saturation of the soil or porous media. The term "coefficient of permeability" is discussed further in section 8.3.2. Hydraulic conductivity is widely used as a synonym for permeability, although there are some theoretical distinctions which have no impact on our analyses. Within this design standard, the two terms will be considered synonymous.

8.1.6.6 Anisotropy

Anisotropy refers to the directional variability of permeability in a given material that is otherwise homogeneous. In seepage through soil, for example, it typically refers to the permeability parallel to bedding or depositional planes (horizontal direction) being different from that in the vertical direction. It can be expressed as a ratio k_H/k_V or k_V/k_H.

8.1.6.7 Saturated Flow

Saturated flow refers to flow in the zone of positive pore pressure beneath the phreatic surface. Saturated flow is produced primarily by a gravity-induced hydraulic gradient between reservoir and exit area (drain or open surface).

8.1.6.8 Unsaturated Flow

Unsaturated flow refers to flow in the zone of negative pore pressure and partial saturation above the phreatic surface. It is produced primarily by a difference in capillary surface tension that induces a hydraulic gradient between the saturated zone and exit area (drain or evaporation face).

8.1.6.9 Transient Flow

Transient flow conditions refer to seepage that is not constant but is changing with time. During transient flow, both pore pressures and effective stresses vary with time. Examples include surface infiltration, an advancing saturation front during first filling, or a drawdown of the reservoir.

8.1.6.10 Seepage Force

Seepage force is that force (acting in the direction of flow) which is exerted on soil particles by the movement of water.

8.1.6.11 Desiccation

Desiccation is the drying of a soil, especially as it relates to an embankment dam and the consequent potential for shrinkage cracking.

8.1.6.12 Failure Mode

A failure mode is a plausible mechanism by which a dam might fail, described in sufficient detail to ensure a thorough understanding of the entire failure process.

8.1.6.13 Internal Erosion

Internal erosion, as used by Reclamation, is a broad or generic term to describe the erosion of particles by water passing through a body of soil; it can include various mechanisms such as piping, scour, etc., as defined later.

8.2 Seepage-Related Issues and Failure Modes

8.2.1 General

Flow of water through soil can lead to movement of the soil grains. Continued movement is erosion. Many factors affect whether soil grains will move or not, including hydraulic gradient, soil plasticity, particle size, capillary tension, cementation, and others. The potential for soil erosion is discussed further in subsequent sections.

A review of historical dam failures indicates that nearly half of all failures of embankment dams have been a result of seepage-induced internal erosion [1]. Additional failures due to high pore pressures or saturated slopes, both attributable to seepage water, add to the list of seepage-related historical failures. Since all soils are erodible to some extent, embankment dams are potentially susceptible to failure due to seepage. In order to evaluate new or existing dams, with respect to safety against seepage, and design defensive measures to mitigate the effects of seepage, it is important to understand the various modes of failure that can occur due to reservoir seepage acting on an embankment or its foundation.

Of further note is that the vast majority of Reclamation's inventory of embankment dams was designed before the failure of Teton Dam. These dams generally do not feature many current state-of-the-practice design features such as internal chimney filters, well designed drains, certain foundation treatment measures, and multiple lines of defense. As such, it is important to carefully evaluate the potential for seepage-related failure modes as we evaluate the safety of our embankment dams.

8.2.2 Excessive Exit Gradients and Uplift Pressures

8.2.2.1 General

Evaluation of seepage forces and pore pressures at the downstream slope and toe area of an embankment is complicated and requires a careful consideration of the site conditions. Simplified formulae for estimating exit (near vertical) gradients and safety factors can be easily misused without an understanding of the specific "failure mechanism" being considered. Furthermore, for a condition at the downstream toe to progress and develop into an internal erosion mechanism that results in a breach of an embankment requires additional considerations. The following sections of this design standards chapter discuss the issues and factors involved in evaluating the criticality of seepage conditions at the downstream toe of an embankment.

An important point to keep in mind when addressing exit gradients or uplift pressures is that the flows and gradients under consideration are typically perpendicular to the embankment slopes or ground surface. Internal (horizontal) gradients, which are typically associated with piping or internal erosion potential along a more horizontal path, are distinctly different from vertical exit gradients and are discussed separately in section 8.2.3.

8.2.2.2 High Exit Gradients in a Cohesionless Soil

In seepage analysis of dams, exit gradients refer to hydraulic gradients at a free face or into more pervious materials. In the case of high upward exit gradients in cohesionless soils, such gradients may result in soft, "quick" ground conditions at the location of the seepage, and perhaps by the presence of sand boils. In a cohesionless foundation soil with a narrow distribution of fine sand and silt grain sizes, a mass of soil can become fluidized as the reservoir reaches the hydraulic head necessary to produce the critical gradient (defined in next paragraph) of the soil mass. A catastrophic slope failure of an embankment can then result if the loss of shear resistance in the soil mass is widespread. In a cohesionless foundation soil with a high percentage of larger particle sizes (medium to coarse sand and gravel), the fine particles in the soil may be removed and deposited on the surface as a "sand boil," while the structure of the large particles remains stable, resulting in an increase in seepage flow. Section 8.2.2.4 provides additional discussion on how these high gradients can lead to dam failure.

An evaluation of high, or critical, (vertical) exit gradients should be limited to conditions of foundation seepage in cohesionless soils. Although this issue is more likely to be of concern in *pervious* cohesionless soils, permeability is less important than the soil cohesion when considering whether to use classical methods of evaluating critical exit gradients. Traditional soil mechanics or seepage discussions on critical exit gradients (such as those by Terzaghi and Peck [2], and Cedergren [3]) have typically only dealt with examples using sand foundations. Determination of critical gradients for a soil results from an evaluation of effective stress conditions. In essence, the critical gradient occurs when the effective stress is zero. Under this condition, a "quick" condition exists in the cohesionless soils, and the foundation materials may "boil" or "heave." Sand boils are a manifestation of localized areas where the critical exit gradient has been reached. The critical gradient (I_c) is most commonly expressed as the ratio of the buoyant unit weight of the soil (γ_b) to the unit weight of water (γ_w):

$$I_c = \gamma_b / \gamma_w$$

An alternate form of this equation, *assuming the foundation soil is saturated*, utilizes the specific gravity (G) and the void ratio (e) of the soil:

$$I_c = (G-1)/(1+e)$$

> **Note:** It is important to recognize that the critical exit (vertical) gradient and the occurrence of boils and heaving of grains only occur in cohesionless soils. In most cohesive soils (plastic clays), with the exception of dispersive soils, inter-particle attractions create bonds between particles that make it less likely for these soils to lose strength due to seepage or for individual particles to be easily moved. Laboratory tests have shown that while sands can typically move or become quick under an upward gradient of around 1.0, clay particles may not move until threshold gradients reach values in the tens or even hundreds. Thus, any type of critical gradient in cohesive soils would be difficult to measure, would vary widely among such soils (due to such variables as percentage of clay fines, type of clay minerals, water content, and density), and should definitely not be calculated by the above equation.

For the case of cohesionless soils, the factor of safety (FS) with respect to vertical exit gradients (against boiling or heave) is generally defined as the ratio of the critical gradient (I_c) to the predicted or measured exit gradient (I_e):

$$FS = I_c/I_e$$

> **Note:** This is **not** a factor of safety against the initiation of internal erosion or the creation of an unfiltered exit. Even if the safety factor is low, or even below unity, there is no guarantee that an internal erosion failure mechanism will initiate or progress. There are many other factors such as the horizontal gradient along the seepage pathway, presence of roof support, and the potential for self-healing to develop that could prevent internal erosion from initiating or progressing. This safety factor simply gives the indication of whether sand boils or heave is probable.

The value of I_e is typically determined by seepage analyses for dams with no piezometric data or by evaluating piezometric data at existing dams, if available. Depending on the state of knowledge about a given site condition, there can be significant uncertainty with the estimated values of gradients (and the resulting calculated factor of safety). Heterogeneous foundation soils can complicate the estimate of the critical gradient. Insufficient numbers of instruments (piezometers) at the downstream toe can lead to an inability to accurately measure actual exit gradients. For new facilities or untested conditions, seepage models may be the only basis for estimating exit gradients. The difficulties associated with modeling natural foundation soils and accurately assigning permeability values and other engineering properties definitely lead to uncertainties in the calculation of gradients. For these reasons, a conservative factor of safety should be used when assessing any threat of high exit gradients. For analyzing existing facilities, a safety factor of 3.0 is considered reasonable, particularly if the structure has performed satisfactorily under near normal loading conditions. However, a safety factor of 4.0 is recommended when designing either a new dam or remedial repairs at an existing dam to rectify a high exit gradient situation. For all cases, if foundation soil properties are well understood and a sufficient piezometer array is available to measure pressures, less uncertainty exists and a lower factor of safety (on the order of 2.0 to 2.5) may be acceptable.

Recommended Factors of Safety Against Heave

Type of Facility	Recommended Safety Factor
New dam	4.0
Existing dam	3.0

8.2.2.3 Uplift of a Confining Soil Layer

If a relatively pervious soil foundation (such as sand) that is not cut off upstream is overlain by a confining layer that is much less pervious (such as clay), dangerously high pressures may exist in the pervious layer. At the downstream toe of an embankment, if the seepage pressures in the pervious layer are higher than the overburden pressure of the confining layer, uplift of the confining layer may occur. A rupture (or "blowout") of the confining layer leads to an exit gradient condition in the pervious layer, which can lead to quick conditions and sand boils as described above. Generally, with this type of situation, the confining layer consists of fine-grained soils with a degree of cohesion and is frequently comprised of clays. Since the concept of critical gradient does not apply to cohesive soils, a different method of calculating the factor of safety against uplift is required.

> **Note:** To reinforce this point, the concept of critical exit (vertical) gradient and "heaving of grains" or "boiling" only applies to cohesionless soils. If the confining layer to be analyzed consists of cohesive soils, a critical exit gradient approach is not applicable to the evaluation of uplift or "blowout" evaluations.

Practitioners may be familiar with the terms "total stress method" and "effective stress method" as means of evaluating uplift of confined layers. Textbooks and literature are not always completely clear in defining whether one method or the other is preferred, or even in distinguishing between the two approaches. Sometimes, it appears that simplified assumptions are made to estimate the potential for uplift without a complete knowledge of the characteristics of the confining layer. In reality, the evaluation of uplift is a rather complicated problem, and frequently sufficient information is not available to get an accurate estimate of the factor of safety against uplift. Appendix A includes a detailed discussion of the difference in methods, as well as examples of how to calculate uplift factors of safety.

> **Note:** This is **not** a factor of safety against the initiation of internal erosion or the creation of an unfiltered exit. Even if the safety factor is low, or even below unity, there is no guarantee that an internal erosion failure mechanism will initiate or progress. This safety factor simply gives the indication of the potential for the confining layer to experience uplift or blowout.

In simplest terms, the factor of safety against uplift can be calculated in total stresses (or forces) as the total downward pressure exerted by the weight of the confining layer divided by the upward water pressure at the base of the layer. This factor of safety by the total stress method is defined as:

$$FS = (\gamma_t)(t) / (\gamma_w)h_p$$

where:

γ_t	=	the total unit weight of the confining layer soil
t	=	the vertical thickness of the confining layer
γ_w	=	the unit weight of water
h_p	=	the pressure head at the base of the confining layer (or top of the pervious layer)

This situation is shown in figure 8.2.2.3-1, which is a part of appendix A, in which the calculations are shown in detail.

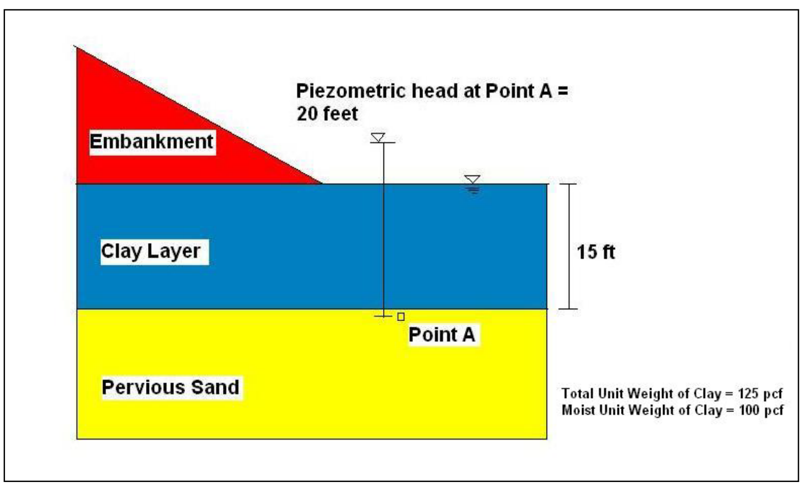


Figure 8.2.2.3-1. Confined layer at downstream toe of embankment.

The so-called "effective stress" approach uses buoyant forces and seepage forces. (Note that both this approach and the preceding method are described well in chapter 17 of Lambe and Whitman, 1969 [4]). With the consideration of seepage forces, the uplift pressure is essentially the differential in total head at the top of the pervious sand layer. The typical equation used for this case is:

$$FS = (\gamma_b)(t) / (\gamma_w)(\Delta h)$$

where:

- γ_b = the buoyant weight of the confining layer soil
- t = the vertical thickness of the confining layer
- γ_w = the unit weight of water
- Δh = the differential piezometric head acting at the base of the confining layer (or top of the pervious layer)

Although the effective stress at the base of the confining layer calculated by these two methods is identical, the approaches give different safety factors against uplift. This difference is illustrated in detail in appendix A. The discussion in appendix A also points out how differences in the phreatic level (or saturation line) within the confining layer can affect the safety factors against uplift for both methods.

Based on the examples in appendix A, the buoyant weight/seepage force (effective stress) safety factor appears more volatile (factor of safety changes dramatically) compared to the total stress method. The effective stress method also appears to indicate safety factors (FS = 3.0 for the example in appendix A) that generally appear somewhat higher than one would expect from most geotechnical engineering analysis cases. In other words, a safety factor of 3.0 would suggest extreme stability in an analysis of static stability, whereas the portrayed example of a blowout situation does not appear nearly so obviously stable. For this reason, Reclamation recommends the use of total stresses for the evaluation of uplift safety factors.

In terms of allowable factors of safety for existing or proposed conditions, it is worth noting some of the uncertainties involved in uplift computations. Higher uplift pressures in the pervious sand layer generally indicate an upward gradient of seepage through the upper saturated clay layer. If there is no upward seepage within the confining layer, or possibly even downward seepage forces in the confining layer due to the presence of a separate source of water (surface seepage, toe drain, or stilling basin "tailwater"), this condition would improve the clay layer's ability to withstand uplift; a benefit that is not explicitly considered in the above calculations.

An additional factor not explicitly accounted for in uplift computations is the actual shear strength or cohesion of the confining layer, particularly if the layer is clayey. For high seepage uplift pressures to cause a rupture in (or even to lift) a clay layer likely takes more than simply exceeding the weight of the layer; those pressures must also overcome the cohesive strength of the clay layer. Lacking any meaningful or efficient way of accounting for this factor, most evaluations simply discount the benefit of this factor in reducing the potential for an uplift failure.

Given these two considerations, uplift computations may underpredict the actual factor of safety against uplift of a cohesive confining layer. However, as discussed earlier for the critical exit gradient issue, reliable data are often lacking to accurately define piezometric levels. For the case of uplift of a confining layer, it is beneficial to have piezometers in both the pervious underlying layer, as well as the confining layer. For most conditions involving existing sites, a reasonable factor of safety using the recommended total stress method can usually be considered 1.5, which allows some margin of error for incomplete piezometric data and variable foundation conditions and soil properties. If piezometers are suspect, or there are insufficient numbers of them, a higher factor of safety should be considered. For new embankment dams, every effort should be made to eliminate uplift or to minimize uplift to achieve a factor of safety of 2.

Recommended Factors of Safety Against Uplift

Type of Facility	Recommended Safety Factor
New dams	2.0
Existing dams	1.5

Note that this type of analysis is frequently performed for new structures or for existing structures under flood conditions not experienced in the past; in either case, the performance has not been tested. Satisfactory performance or acceptable safety factors under existing conditions, or past loadings, does not guarantee safe conditions under higher pressures.

8.2.2.4 Implications of High Vertical Exit Gradients and Uplift Pressures

If analyses described above indicate the potential for seepage gradients to approach the critical gradient or for uplift pressures to be near the resisting overburden pressures, it is possible that the embankment and foundation may experience sand boils (in a cohesionless foundation) or, possibly, cracking of a low permeability confining layer. Perhaps the most obvious failure mechanism is that these events will then lead to progressive backward erosion and ultimate dam breach. It is important to note that laboratory tests by Schmertmann [5] and Townsend et al. [6] led those researchers to conclude that the presence of low or zero effective stress conditions at the exit point (due to high exit gradients) helps promote the initiation of backwards erosion piping. On a micro scale, it appears that very small slope failures or collapses of the “pipe” walls occur at the exit point and continue to progress upstream. In the opinion of these researchers, a condition of zero strength at the exit point may help explain why horizontal piping can occur at quite low hydraulic horizontal gradients (discussed in the next section) in soils with very low plasticity.

High pore pressures can also lead to embankment instability as discussed in section 8.2.4.

8.2.3 Unfiltered High Internal Gradients

8.2.3.1 General

Whereas the previous section on exit gradients dealt primarily with vertical gradients, this discussion focuses on internal gradients through an embankment or foundation, which are generally horizontal or often nearly so in many (if not most) cases of internal erosion failure mechanisms. Although formulae exist for computing factors of safety for conditions of critical exit (vertical) gradients, there is much more uncertainty when it comes to determining internal (horizontal) gradients that are capable of initiating internal erosion. This uncertainty comes from the variables and differing conditions inherent in lengthy seepage flow paths through embankment or foundation soils. It is possible to estimate an overall average gradient for a seepage path given an upstream piezometer and a downstream piezometer (or even using reservoir level and tailwater). However, the internal gradients are likely quite different at various places along the seepage pathway since natural, or even engineered, soils can be highly variable. The seepage path is undoubtedly not a straight line and likely meanders considerably, with seepage flows experiencing different amounts of head loss at different locations along the way. It is extremely unlikely that sufficient piezometers would be located in a number of critical locations along a seepage pathway in or beneath a dam to accurately measure the piezometric pressures at key points in a critical (weak link) flow path. Furthermore, it is exceedingly difficult to accurately assess how the soils along an entire seepage pathway will respond to seepage gradients. Laboratory tests can provide insights into how a relatively small segment of representative soil will behave under various hydraulic gradients, and these studies suggest that key factors like soil plasticity and grain size are important parameters in determining the potential for internal erosion. In actual field conditions, both soils and gradients are expected to vary in most instances.

These complex variables, as well as many other physical or chemical factors which play a role in an internal erosion process, help explain why there is no widely accepted means to determine the factor of safety against internal erosion or piping. Rather than using deterministic safety factors, Reclamation practitioners typically use available laboratory testing, research, and empirical evidence to probabilistically estimate internal erosion potential in risk analyses. References to consider in aiding these determinations include research from the University of New South Wales [1], the work by Schmertmann and by Townsend et al. [5, 6], and the section on "Internal Erosion Risks" in the *Best Practices in Dam and*

Levee Safety Risk Analysis training manual prepared by Reclamation and the U.S. Army Corps of Engineers [7]. A particularly important finding from the research by Schmertmann and Townsend has been the determination of a minimum gradient for internal erosion in clean, fine, uniform sands. Based on their laboratory tests, this type of sand was found to experience internal erosion (backward erosion piping) at a gradient as low as 0.08. This appears to be modeling a fairly severe scenario, in that a highly erodible soil was used and a roof (consisting of a plexiglass plate) was formed above it. In most field conditions at Reclamation facilities, coarser soils will likely be encountered, and a natural roof may not form along the entire seepage path. However, it should be pointed out that the piping experienced at A.V. Watkins Dam in 2006 involved materials and conditions quite similar to the model used by Schmertmann and Townsend, and the piping is believed to have occurred at a low gradient similar to that measured by those two researchers. An additional case history is Wister Dam in Oklahoma, where internal erosion may have occurred under gradients as low as 0.02 in the embankment comprised of dispersive clays, materials that are typically highly erodible, and that go into suspension in the presence of water [8]. In the case of Wister Dam, the internal erosion probably occurred as a result of cracking due to differential settlement and/or possibly hydraulic fracturing, rather than piping.

> **Note:** It is worth reinforcing the concept that the internal gradient that might lead to the initiation of internal erosion may be as low as 0.02 to 0.08 for particularly susceptible soils. These internal (horizontal) gradients are much lower than the "rule of thumb" critical gradient of 1.0 often assumed for exit (vertical) gradients.

When considering the potential for internal erosion initiating due to high internal gradients, it is helpful to have an understanding of the specific types of failure mechanisms that might occur. The following sections discuss those mechanisms typically evaluated in Reclamation studies.

8.2.3.2 Classical Piping

Classical piping occurs when soil erosion begins at a seepage exit point and erodes backwards through the dam or foundation, with surrounding soil providing a support (roof) to keep the developing pipe open. Four conditions are needed for development of piping: (1) a concentrated leak/source of water (of sufficient quantity and velocity to erode material), (2) an unprotected seepage exit point, (3) erodible material in the flow path, and (4) material being eroded or material adjacent to it capable of supporting a pipe or a roof [9]. If these conditions all develop, uncontrolled erosion begins to create a pipe within the embankment or foundation. Ultimately, the pipe could progress to the upstream slope and lead to gross enlargement of the erosion pathway and dam breach, or the pipe could lead to the formation of a large sinkhole in the dam that collapses and leads to crest loss and dam overtopping. An additional type of breach could result from the erosion leading to oversteepening of the downstream slope and progressive sliding or sloughing that ultimately advances back through the crest and leads to overtopping. Additional discussion of some considerations for whether or not an internal erosion failure mode will progress to dam breach is included at the end of Appendix A.

In Fell et al., 2008 [10], backward erosion is termed as a special case of internal erosion initiating at the exit point where the material being eroded is cohesionless and cannot support a roof, but there is overlying material capable of forming a roof. It is believed that this latter type of soil and condition is particularly susceptible to the initiation of internal erosion. Reclamation has had several instances where backward erosion or piping in low plasticity soils has led to a significant internal erosion event. Key instances include internal erosion in the foundation of A.V. Watkins Dam and internal erosion along the Caldwell Canal outlet works conduit at Deer Flat Dams. It is important to note that this type of internal erosion is not always rapid but can be gradual, taking decades instead of hours or days. In some cases, the internal gradients are only at critical levels for short periods of time, and erosion is thus intermittent or episodic.

8.2.3.3 Internal Migration

Internal migration can occur when the soil is not capable of sustaining a roof or a pipe. Soil particles are eroded, and a temporary void grows until a roof can no longer be supported, at which time the void collapses. This mechanism is repeated progressively until the void shortens the seepage path and leads to uncontrolled erosion and, ultimately, to breach of the dam as discussed in section 8.2.3.2. An internal migration event often manifests as sinkholes and stoping of embankment materials. Examples of Reclamation incidents include the sinkholes at Willow Creek Dam (Montana), Helena Valley Dam, and Davis Creek Dam.

8.2.3.4 Scour

Scour occurs when tractive seepage forces along a surface (e.g., a crack within the soil, adjacent to a wall or conduit, or along the dam/foundation contact) are sufficient to move soil particles into an unprotected area. Once this begins, failure from a process similar to piping or progressive erosion could occur. Suspected examples of scour at Reclamation facilities include the internal erosion events at Steinaker and Fontenelle Dams, as well as the failure of Teton Dam.

8.2.3.5 Internal Instability, Suffusion, and Suffosion

Suffusion and suffosion are internal erosion mechanisms that can occur with internally unstable soils. The two processes are similar, and there are some inconsistencies in international published literature with their definitions and descriptions.

Suffusion: A form of internal erosion which involves selective erosion of finer particles from a matrix of coarser particles that are in point-to-point contact in such a manner that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles. Suffusion typically involves little or no change in volume of the soil mass.

Suffosion: A form of internal erosion which initially involves selective erosion of finer particles from a matrix of coarser particles, but at higher gradients also involves movement of medium size particles. The volume of finer particles is such that the coarser particles are "floating" and not in point-to-point contact. Suffosion involves a decrease in volume of the soil mass.

With suffusion, seepage erosion of the finer fraction through the coarser skeleton will result in a higher permeability material and no volume change in the material being subject to suffusion. Suffusion of core materials or foundation materials (typically, glacial foundation materials) could result in increased permeability and increased seepage flow (and possibly velocity). As seepage flow velocity increases, another internal erosion mechanism (i.e., scour) could initiate.

Suffosion differs from suffusion because the coarser materials are not in point-to-point contact and, therefore, when they erode out, volume change is observed (e.g., sinkholes, depressions). The velocity of flow through the soil must impose high enough stress to overcome the stresses imposed on the particles by the surrounding soil. Compared to suffusion, suffosion requires a more extreme combination of seepage velocity and gradient to initiate particle movement because the in situ stresses imposed on the finer fraction particles by the surrounding soils must be overcome for erosion to occur.

Suspected Reclamation examples of suffusion/suffosion include the incidents at Bumping Lake, Keechelus, and Twin Lakes Dams (all founded on glacial soils).

8.2.3.6 Analysis of Unfiltered High Internal Gradients

Although it is difficult to predict and determine actual internal gradients in existing dams, the use of numerical models can help estimate such gradients in both existing dams and new dams. These methods are discussed beginning in Section 8.4, "Seepage Analysis Principles and Procedures." Section 8.4.6 details the use of computer programs to calculate important aspects of seepage behavior.

Data obtained from numerical seepage analyses can include estimated pressures and piezometric heads at various points within the embankment model, from which gradients can be calculated. For existing dams, piezometers within an embankment and/or foundation are sometimes used to calculate internal gradients as well. Frequently, such gradients determined from piezometers or seepage models will exceed the minimum of 0.08 determined in the Schmertmann and Townsend research on clean fine sands [5, 6]. Whether or not such gradients are potentially harmful is generally a matter of engineering judgment and consideration of a number of additional factors such as soil gradation and plasticity, filter compatibility with adjacent soils, potential for self-healing, dispersiveness, etc. For Reclamation dams, the potential for erosion to initiate is typically evaluated by means of risk analysis of various internal erosion failure modes. A useful reference for this process is the chapter on internal erosion in the *Best Practices in Dam and Levee Safety Risk Analysis* training manual [7].

An admittedly simplified means of considering whether internal horizontal gradients may be of concern is the "weighted creep ratio," based on work originally developed by Bligh in 1910 and subsequently modified in 1935 by Lane (who investigated more than 250 masonry dams on soil foundations, of which more than 20 involved a failure). The development of this concept is discussed well in Lane's original paper [11] and in Terzaghi and Peck [2, pp. 615-618]. In essence, empirical evidence was assembled to demonstrate when piping was unlikely to occur in various types of soils. The weighted creep method suggests how long a seepage path must be in various soils before reaching the threshold of potential internal erosion. The use of this method in very fine sands or silts indicates that the safe (horizontal) gradient to prevent the development of internal erosion is approximately 0.04 for a worst case assumption of solely horizontal flow to an open exit. Critical gradients increase for larger sized soils. The weighted creep method also demonstrates the importance of any vertical flow components in reducing the potential for internal erosion to occur. This method is no longer used; the typical practice is instead to perform seepage analyses for critical or large structures such as dams. However, Lane's method is based on a significant amount of empirical data, and a key contribution is its demonstration that internal erosion can, in fact, initiate at relatively low horizontal gradients.

Given the uncertainties in evaluating the potential for internal erosion due to unfiltered high internal gradients within an embankment or its foundation, it is prudent to ensure that all failure modes are carefully considered. Special caution should be exercised for situations similar to case histories where internal erosion has initiated. Critical situations include, but are not limited to, the following:

- Seepage through soils with low or no plasticity

- Dispersive clay soils or similarly highly erodible soils

- Seepage through the upper portion of any embankment that may have experienced cracking due to differential settlement, desiccation, or other causes

- Seepage into or along outlet works conduits or similar penetrating features

- Seepage along a contact between a soil and a concrete wall, particularly if the wall is vertical

- Seepage into inadequate drainage features such as toe drains and appurtenant structure underdrains
- Dams with adjacent zones of obvious filter incompatibility (includes dam/foundation contacts)
- Dams with fractured bedrock foundations that did not receive thorough foundation treatment during construction
- Seepage paths that are essentially horizontal with little vertical component or obstruction to the pathway
- Dams without fully penetrating foundation cutoffs

8.2.3.7 Design Details to Protect Against High Internal Gradients

All new dams or modifications to existing dams shall incorporate design details to ensure that any seepage under high internal gradients is filtered prior to exiting to a free face, or nonerodible features are included to reduce gradients. The primary defense against internal erosion is the use of filters. Filters shall be carefully designed and constructed to ensure that seepage through erodible soils does not have an open or unfiltered exit such as coarser soils or rockfill (whether in the foundation or adjacent embankment zones), bedrock fractures or apertures, weep holes or cracks in concrete structures, open drains, etc. All potential seepage exits shall be properly filtered to ensure that no particle transport can initiate. Other defenses utilized against high internal gradients include: (a) careful foundation treatment at the embankment/bedrock contact; (b) use of plastic soils in embankment cores; (c) use of wide cores and fully penetrating cutoff trenches; (d) the use of upstream blankets or cutoff walls to lengthen the seepage path; and (e) grouting of the foundation to seal geologic defects and reduce seepage. Protective measures to defend against seepage are discussed further beginning in Section 8.5, "Seepage Mitigation Measures." Additional references include Reclamation's *Design Standard No. 13, Embankment Dams*, Chapter 2, "Embankment Design," and Chapter 5, "Protective Filters."

> **Note**: Many, if not most, Reclamation dams do not include modern defenses against internal erosion; thus, they must be carefully evaluated for potential seepage-related failure modes.

8.2.4 High Pore Pressures and Seepage Forces

Uncontrolled or poorly controlled seepage can lead to the formation of high pore pressures and/or high seepage forces in an embankment dam or its foundation. High pore pressures at the toe area of a dam can lead to quick conditions or uplift concerns in foundation soils as discussed in section 8.2.2. However, a typical concern with these types of pressures is the impact on overall embankment slope stability. The presence of high pore pressures in the embankment or foundation results in a decrease of effective stress/strength in those soils, which makes the embankment more vulnerable to a slope failure. Similarly, seepage passing through an embankment generates seepage forces acting along the flow lines. For seepage exiting at or above the downstream toe, these seepage forces add to the driving forces and reduce the safety factor against stability. In some cases, failure results not from a deep-seated slope failure, but from progressive sloughing of the downstream slope. This is a key reason that embankments are zoned or contain internal filters and drains—to ensure that the downstream portion of the embankment is outside the seepage zone (remains unsaturated and strong) and can provide support for the central portion of the dam.

Although this design standard concentrates on embankment dams, it is important to recognize that high pressures can also severely damage other structures at an embankment dam facility. Examples include:

- Uplift of spillway chutes, stilling basins, and canal linings
- Sliding failures of gravity dams or walls due to high pressures at the base
- Retaining wall failures due to high hydrostatic pressures

The primary defense against high pressures is the inclusion of well-designed filter/drainage systems to prevent the formation of high pressures due to seepage.

8.2.5. Excessive Seepage Flows

Excessive seepage flow without soil erosion (such as described in the next section) is usually not a structural or dam safety related failure mode; however, it could be considered a project failure if it results in a serious loss of project water and benefits. Downstream flooding or destructively high ground water levels could also result from excessive seepage. In addition, the uncertainty over the extent and threat of seepage during inspections and analyses, as well as the public perception of a leaky dam, are potential concerns. Furthermore, with high seepage flows, a developing internal erosion condition may develop fairly quickly. For these reasons, designs shall include features to minimize and control seepage flows.

8.2.6 Dissolution and Karst

Some bedrock foundations, including limestone, gypsum, and anhydrite, are soluble and are subject to dissolution by reservoir seepage. Although limestone dissolution is generally considered in terms of centuries, more soluble rocks like gypsum and anhydrite can dissolve in tens of years. Dissolution of foundation bedrock can create several potential failure modes, including the formation of open pathways in the foundation into which embankment materials can erode; high seepage flows through the upper portion of the rock, which lead to scour erosion at the base of the embankment; or the development of high pore pressures in the foundation or lower portion of the embankment, which can lead to stability concerns.

Even if dissolution during the life of the reservoir does not occur to a significant extent, it is possible that the bedrock has experienced dissolution in past "geologic time." The presence of solution channels and ancient collapsed sinkholes in soluble rocks creates what is referred to as "karst" conditions. Seepage flowing through these features can result in the same failure modes as discussed in the preceding sections of this design standard. However, rather than developing through dissolution due to reservoir-induced seepage, existing seepage pathways are more likely to be enlarged by erosion and removal of soil infillings within the ancient karstic channels. Examples of dams experiencing internal erosion due to soluble foundations include Wolf Creek Dam in Kentucky and Mosul Dam in Iraq.

Given the unpredictability of natural dissolution patterns, it is difficult to drill enough holes to fully evaluate the risks associated with a karstic foundation. Hence, extreme caution is required when evaluating or designing a dam on a foundation which includes soluble rocks. For an existing project, water quality tests taken over a period of years can indicate whether dissolution (or infilling erosion) is occurring by monitoring the amount of certain dissolved minerals and compounds (or suspended particles) in seepage waters. Geophysical testing to detect seepage pathways can also be used some cases. Monitoring of foundation piezometers and surface seeps can also provide information as to whether the foundation conditions are changing. For a new dam, extensive foundation exploration is usually required, and foundation treatment measures will typically include considerable grouting and, potentially, the inclusion of a cutoff wall through the soluble portions of the bedrock. If these features are inadequate on existing dams, it may be necessary to add them.

8.2.7 Hydraulic Fracturing

Hydraulic fracturing refers to fracturing or cracking induced in a soil when hydraulic pressures acting on the soil exceed the minor principal stresses and tensile strength of the soil. This topic is covered particularly well in a 1985 paper by Sherard [12]. Hydraulic fracturing most typically occurs within an embankment during improper drilling or grouting operations. Since pressurized fluids are used in these operations, it is possible to create a situation where high-pressure fluid exists in a drill hole, with pressures that exceed the stresses in the dam which are a function of the height of the fill. Thus, particularly vulnerable locations within a dam are where existing confining stresses are anomalously low (such as near a vertical contact, near a penetrating conduit, in a narrow trench, etc.) The high hydraulic pressures force open a crack in the embankment, which could theoretically propagate for a significant distance. Although, in many cases, the crack may be filled with the drilling mud or grout used in the triggering operation, there is the potential that an untreated defect could be formed within the embankment core. For this reason, drilling and grouting operations through the core of a dam are generally avoided or undertaken only with careful provisions to minimize the potential for hydraulic fracturing and preventing compressed air or foam [13].

Another vulnerable condition in an embankment that may lead to hydraulic fracturing is any area of low stress, particularly areas where "arching" may lead to portions of the embankment not experiencing the full weight of the overlying soils. An obvious example of this condition would be in a narrow, steep-sided excavation, perhaps a cutoff/key trench or an excavation for an outlet works conduit. For these types of features, overlying soil is able to arch or bridge over the narrow excavation, resulting in relatively low stresses in the backfilled trenches. If reservoir water has access to these areas, high water pressures could lead to hydraulic fracturing of the low stress soils. This type of mechanism has been suggested to possibly have played a role in the failure of Teton Dam, which featured a very narrow, steep-sided key trench backfilled with low plasticity soil. Design details to prevent this potential failure mode include the use of wider excavations with flatter slopes and the use of filters to ensure that if a crack does form in the compacted embankment, any seepage path is filtered.
Ironically, another potential hydraulic fracturing scenario can occur in a seepage reduction design feature. Slurry trench cutoff walls are vertical trenches kept open by bentonite slurry but, ultimately, backfilled by a variety of materials. This type of vertical trench poses a high potential for arching, and backfill materials such as soil-bentonite are unlikely to experience the full weight of overlying material. Hence, soil-bentonite slurry trench cutoff walls (or walls constructed of similar low-strength backfill) are potentially vulnerable to hydraulic fracturing. Hydraulic fracturing occurred in a soil-bentonite cutoff wall at Reclamation's Virginia Smith (Calamus) Dam while it was under construction. To alleviate this concern on a project where the cutoff wall will be the sole or primary seepage reduction measure, stronger backfills such as concrete (or soil-cement if conditions are favorable for its use) are typically specified.

8.2.8 Desiccation Cracking

Rather than a failure mode resulting from seepage flow, desiccation cracking is instead caused by a lack of seepage flow or moisture in a clay or plastic soil. Specifically, desiccation results from a decrease in the moisture content and subsequent shrinkage cracking. The potential for desiccation increases with increasing soil plasticity. For zoned embankments, desiccation is generally only a concern in the core. Desiccation can result in transverse cracking of an embankment which, in turn, leads to the possibility of internal erosion through the crack, typically in the upper portions of the dam. Some of the causes of desiccation cracking include:

- Evaporation from the ground surface, particularly in arid/hot areas such as the Western United States
- Extended periods of low reservoir levels
- Capillary action in fine grained soils

A method for evaluation or analysis of desiccation cracking is not obvious; rather, embankment designers/analysts simply need to be aware of the need to potentially protect the upper portions of an embankment. The most reliable defense is to include filters that extend to the dam crest to prevent internal erosion through desiccation cracks when the reservoir is subject to extreme flood levels. An example of this phenomenon is illustrated by the State-owned Stubblefield Dam in northern New Mexico. Reclamation designed and constructed a fix for these dams in partnership with the New Mexico State Engineer's Office. The fix consisted of a vertical trench excavated through the dam crest to a depth of around 10 feet, which was then backfilled with sand and gravel filter material.

8.2.9 Differential Settlement Cracking

Cracking can also occur in embankment dams due to differential settlement cracking. In fact, this type of cracking is regarded as much more likely than desiccation cracking [14]. Some of the conditions that could lead to this type of cracking include:

- Wide benches or "stair steps" in the upper to middle portion of the abutment
- Steep abutments near the top of the dam
- Foundation irregularities such as overhangs
- Soft foundation materials that are not removed or treated
- Foundation materials of differing compressibility that can settle unevenly
- Embankment zones of different stiffness that might settle at different rates, leading to "dragging" and "transverse shearing" of the core

8.2.10 Additional Defects

There are other conditions that may result in a “defect” within an embankment that could serve as an avenue for seepage. Such conditions include:

- Embankment cores that were constructed with poor techniques, such as very thick lifts and no or inadequate rollers (lack of formal compaction)

- A coarser lift within the core due to variation in the borrow area or segregation during placement (poor construction techniques)

- A lift corresponding to a winter (or perhaps rainy) shutdown period that was not properly treated or removed before placing additional lifts

- A penetrating feature through an embankment core, such as a conduit or hydraulic piezometer trench, in which improper compaction occurred

- Poor compaction or construction techniques at the contact of the embankment and foundation

8.2.11 Collapsible Soils

Collapsible soils include fine grained loessial soils and fine sands found throughout the Great Plains States and other areas of the country, as well as some desert soils found in the Southwest. These soils are usually windblown deposits. In essence, collapsible soils have been deposited in a very loose state and frequently feature an open structure arrangement of the particles held together by lightly cemented clay bonds. These soils can exhibit relatively high strength and stiffness in a dry state. The presence of water tends to destroy the bonds and can lead to a collapse of the honeycomb structure, resulting in significant settlement. These soils may or may not settle under their own weight upon saturation, but they are far more likely to collapse if loaded.

Thus, embankments and other structures founded on collapsible soils may settle and crack when the foundation becomes wetted for the first time. Such cracking may, in turn, lead to internal erosion, which can quickly lead to dam breach because these soils are typically of low plasticity. Another failure mechanism can occur during ground water drawdown. Since these types of soils may not collapse simply upon saturation, additional loading sometimes triggers the collapse. A drawdown of the ground water level creates a higher load on the foundation soils (because overlying soils become total unit weights instead of buoyant unit weights) and can lead to collapse settlement.

The most common ways to treat potentially collapsible soils include removal and replacement with compacted fill and prewetting and preloading of the soils, although dynamic compaction may be considered for shallow soils. The prewetting technique typically involves wetting the soils by sprinkler systems or ponding, and then loading the soils with soil covers or berms. The intent of this technique is to cause the soils to collapse before construction of the embankment and before reservoir operation. Soils that do not collapse before construction can still collapse as the embankment fill is raised over time. Prewetting has not always worked and could leave a deposit that is vulnerable to seismic liquefaction or settlement. The preferred method is complete removal of the collapsible soils to eliminate the risk of cracking due to collapse (although this may be the most expensive option).

8.3 Key Data for Seepage Evaluations

8.3.1 General

Predicting seepage behavior, like many other tasks in geotechnical engineering, is difficult due to the many variables involved in evaluating and modeling soil materials, as well as potentially complex foundation conditions. In light of these difficulties, it is important to conduct a thorough search and review of available information that may aid in the evaluation and analysis of seepage issues. The following sections of this design standard describe various types of data, investigations, or tests that may provide a better understanding of the site-specific conditions that could impact seepage behavior at a given site.

8.3.2 Permeability

The permeability of soil and rock materials in or beneath an embankment is the most obvious factor that plays a key role in seepage behavior. That being said, it can also be a very difficult parameter to measure, which implies that several methods to estimate permeability may be useful to get an understanding of the potential range in permeability values at a given dam and foundation. Permeability, or more precisely "coefficient of permeability," is at times used interchangeably with the term "hydraulic conductivity." Throughout this design standard, the two terms will be used interchangeably to refer to the flow rate through a saturated porous medium under a unit (1.0) hydraulic gradient. The theoretical basis for determining permeability for laminar flow through the use of Darcy's Law is discussed further in section 8.4.1.

Since permeability is defined above as a flow rate, it is typically expressed in such units as feet per year (ft/yr) or centimeters per second (cm/s). Figure 8.3.2-1 is a chart from Lambe and Whitman [4], which can be used to convert permeability values from one set of units to another.

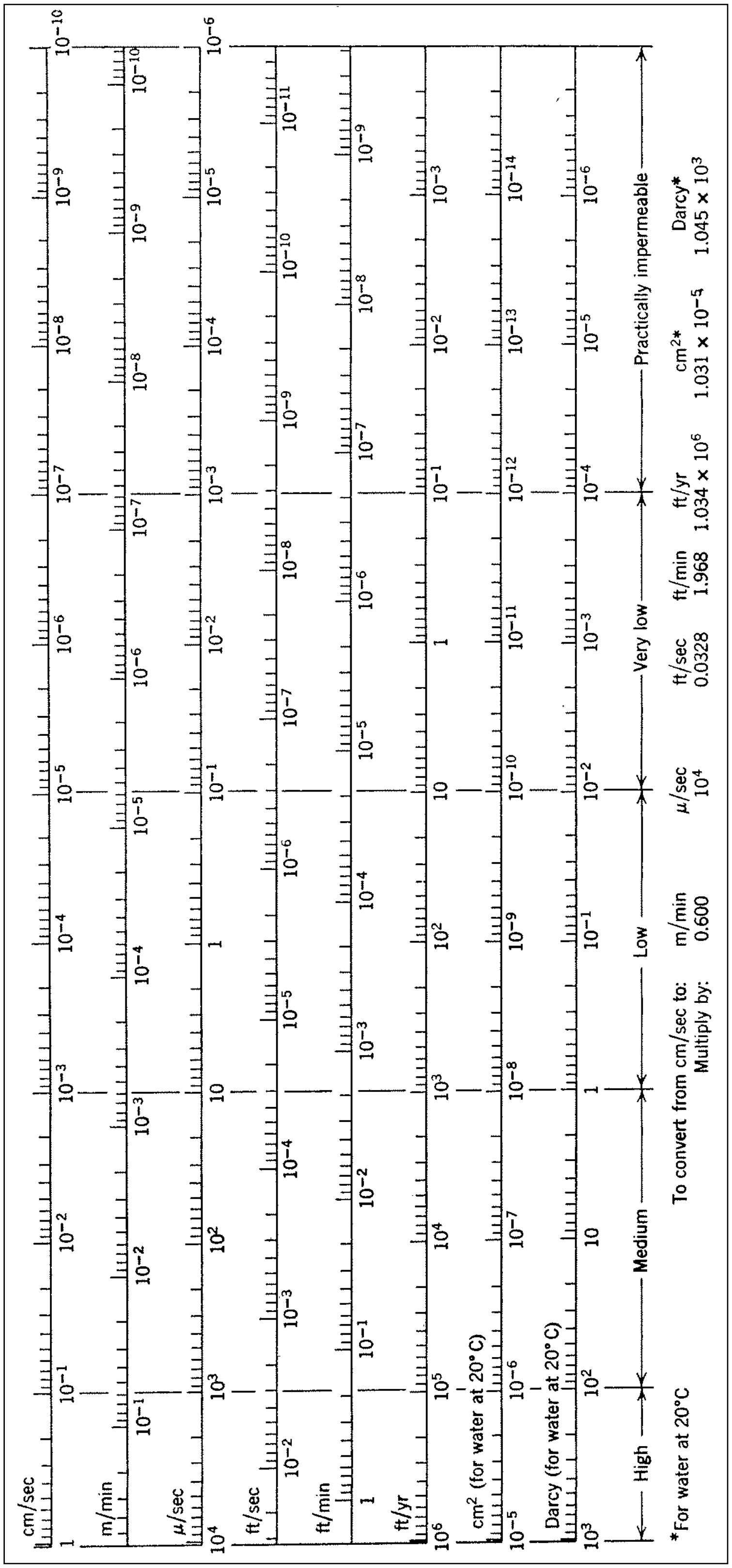


Figure 8.3.2-1. Permeability conversion chart [4].

There are a number of factors which can influence the permeability of a soil, and there are a number of methods which can be used to estimate permeability. The following paragraphs provide a discussion of some of the key factors affecting permeability and how to use past research and studies of these factors to estimate permeability, as well as describe typical field and laboratory tests used to estimate permeability for various soils and rocks.

8.3.2.1 Estimating Permeability from Published Data for Various Soils and Rock

One way to estimate permeability values for soils and rock at a given site is to refer to published information on permeabilities. This type of information can be used for estimates of permeability, for comparison/calibration of existing information for a given dam or foundation, and for initial input into numerical seepage models. There has been a great deal of research on the topic, and researchers have related permeability values to various soil and rock properties. Following are some key discussions and published relationships.

> **Note:** Embankment and foundation soils with the highest permeabilities generally control the results of seepage analyses. Therefore, the greatest focus should be on estimation of the pervious layers at any site. Relatively impervious units (such as clay layers or embankment cores) will typically have permeability values that are orders of magnitude lower and, thus, may not require highly accurate values.

8.3.2.2 Typical Range of Permeability Values in Soil and Rock

It is recognized that this particular property of a soil (permeability) has an extremely wide range of values across soil types. Figure 8.3.2.2-1 is a simple representation of typical permeabilities [15] for various types of soil and rock, which shows a range of values spanning more than 10 orders of magnitude. No other engineering property of soil (or any other construction material, for that matter) has this degree of variability.

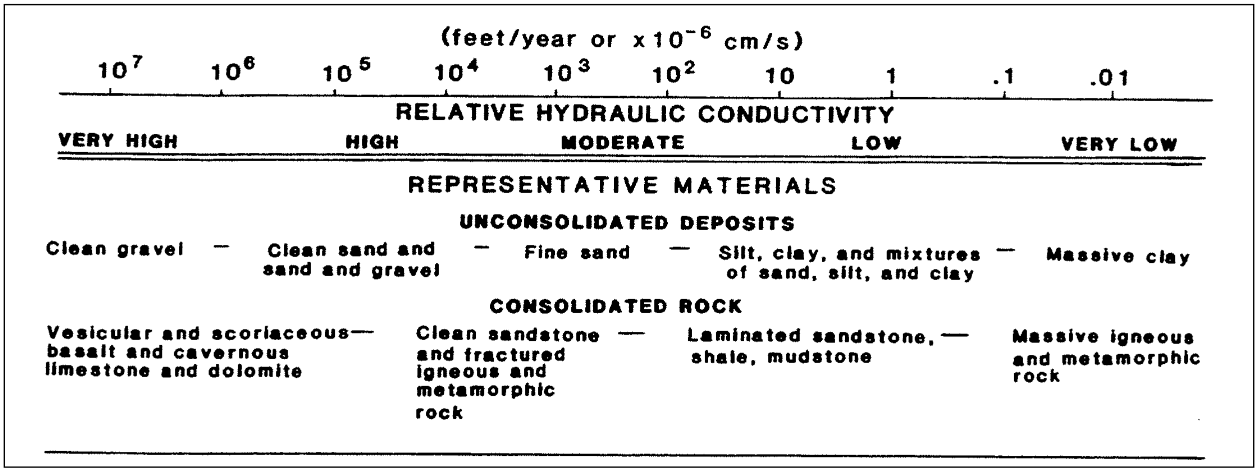


Figure 8.3.2.2-1. Hydraulic conductivities for various classes of geologic materials [15].

8.3.2.3 Variations in Permeability Values Due to Anisotropy

In addition to potentially having a wide range of permeability, most soils can have significant variability in the ratio of horizontal to vertical permeability given the general stratified nature of both natural and, in some cases, man-placed deposits. In this design standard, this difference in horizontal and vertical permeability is defined as anisotropy. Anisotropy is primarily due to the method of deposition or placement but also can be influenced by particle shape and orientation. In rock, the fracture and joint pattern is obviously a key factor.

8.3.2.3.1 Anisotropy in Natural Soil Deposits

Water-deposited soils, which include alluvial/fluvial and lacustrine types of deposition, are typically deposited in horizontal layers and are highly stratified in nature. Such deposits can have horizontal to vertical permeability ratios (k_H/k_V) of more than 100. Fine-grained strata control the vertical permeability, and coarse-grained strata control the horizontal permeability. For a given stratified deposit of significant thickness, a single continuous layer of clay will likely control the overall vertical permeability of the entire deposit, while a uniform, open-work, or particularly permeable coarse layer will likely control the horizontal permeability of the deposit. It thus becomes essential to accurately define the stratigraphy of such a soil foundation.

Windblown deposits such as dune sand and loess tend to have low values of k_H/k_V, typically ranging from 0.2 to 2. (For permeability values in loess, reference [16] contains a summary of a significant amount of testing of loessial soils associated with Reclamation facilities.) These types of deposits are often more permeable in the vertical, rather than the horizontal, direction due to the presence of continuous root holes (and the typical lack of horizontal bedding). As stated earlier, a complication with assessing the permeability of windblown soils is that they can be subject to significant collapse upon wetting, which can lead to significant changes in permeability between the in situ condition and post-wetting (post-reservoir) condition.

Figure 8.3.2.3.1-1 shows some typical permeability values referenced from various sources [15-42] for natural soils, as well as the expected range of anisotropy.

Permeability k_H of Unconsolidated Natural Soils (k_H inversely related to % finer grains)

Soil	k_H Range (ft/yr or 10^{-6} cm/s)
Gravel, open-work	>2,000,000
Gravel (GP)	200,000 to 2,000,000
Gravel (GW)	10,000 to 1,000,000
Sand, coarse (SP)	10,000 to 500,000
Sand, medium (SP)	1,000 to 100,000
Sand, fine (SP)	500 to 50,000
Sand (SW)	100 to 50,000
Sand, silty (SM)	100 to 10,000
Sand, clayey (SC)	1 to 1,000
Silt (ML)	1 to 1,000
Clay (CL)	~0 to 3

References: [15], [18], [22-29], [32-36]

Permeability k_H of Unfractured Rock (k_H increases with pore size)

Rock	k_H Range (ft/yr or 10^{-6} cm/s)
Sandstone, medium	100 to 200,000
Sandstone, silty	~0 to 5,000
Limestone	~0 to 15,000
Granite, weathered	200 to 10,000
Schist	~0 to 2,000
Tuff	~0 to 1,000
Gabbro, weathered	50 to 500
Basalt	~0 to 50
Dolomite	~0 to 5
Gneiss	~0 to 2

References: [15], [25], [27-28], [34], [36]

Anisotropy of Natural Soil and Rock

Formation	k_H/k_V	Remarks
Stratified deposits	10 to 1,000	k_H/k_V depends on grain size of substrata
Massive soil or rock	1 to 3	Depends on particle shape and orientation
Fractured rock	0.1 to 10	Depends on aperture arrangement
Eolian soil (loess and dune)	0.02 to 2	Depends on consolidation

References: [3], [15], [17], [19-21], [24], [30-32], [35-42]

Figure 8.3.2.3.1-1 Permeability of natural soil and rock.

8.3.2.3.2 Anisotropy in Bedrock

Permeability in rock is typically defined in terms of primary and secondary permeability. Primary permeability in rock refers to flow through the grain structure of the material. Secondary permeability refers to flow through joints, fractures, or other finite open discontinuities in the rock unit. Primary permeability in unstratified (massive) permeable rock generally has low anisotropy (k_H/k_V equal to 1 or 2). Fractured rock anisotropy, or secondary permeability, is quite complex and governed by factors such as fracture orientation, fracture density, and aperture size. These factors may also govern whether Darcy flow is assumed to apply to bedrock.

Figure 8.3.2.3.1-1 shows some typical permeability values for rock, as well as the expected range of anisotropy. Figure 8.3.2.3.2-1 is a graph from Morgenstern [43] that relates secondary permeability of a rock mass as a function of aperture size. It is worth noting from this figure that relatively small apertures can result in relatively high permeability values; significant seepage can result from small defects in bedrock or conduits if these defects are continuous.

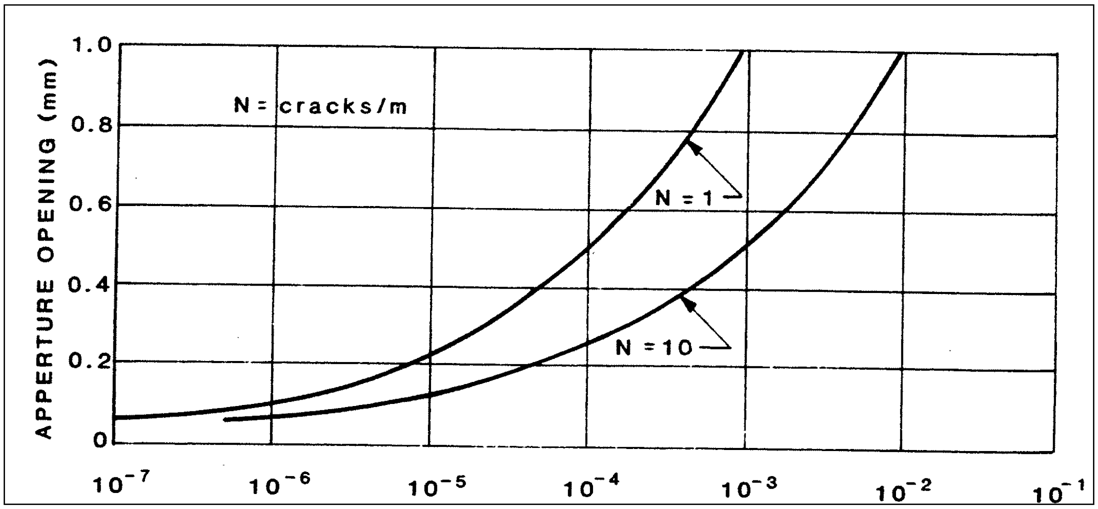


Figure 8.3.2.3.2-1. Equivalent secondary permeability of a simple array of parallel cracks [43].

8.3.2.3.3 Anisotropy in Embankment Soils

Test and performance data from Reclamation embankments indicate that typical dams will have k_H/k_V ratios ranging from 2 to 10, with the higher values relating to higher water contents during placement. Older dams, such as those constructed in the early part of the 20th century or by hydraulic fill methods, may have anisotropy as high as 50 due to stratification during placement and earlier compaction methods that did not emphasize mixing and discing.

However, coarse-grained materials, such as rockfill shells or filter and drain materials, are typically placed in thicker lifts without as much compactive effort, and they tend to have lower anisotropy. These types of soils are often assigned anisotropy values of 1.

Figure 8.3.2.3.3-1 shows some typical permeability values referenced from various sources [3, 17-18, 21, 26, 31-34, 36-37, 44-45] for embankment materials, as well as the expected range of anisotropy.

Permeability (k_V) of Embankment Core Materials (k_V inversely related to % fines)

Unified Soil Classification	k_V Range (ft/yr or x10^{-6} cm/s)*
GM-SM	0.0 to 10.0
GM or GC	0.0 to 10.0
SP-SM	0.0 to 10.0
SM	0.0 to 10.0
SM-SC	0.0 to 3.0
SM-ML	0.0 to 10.0
SC	0.0 to 3.0
ML	0.0 to 10.0
ML-CL	0.0 to 1.0
CL	0.0 to 1.0
MH	0.0 to 0.1

References: [31-32], [34], [44-45]
* Based primarily on Reclamation laboratory test data

Permeability (k_V) of Embankment Shell Materials (k_V inversely related to % fines)

Unified Soil Classification	k_V Range (ft/yr or x10^{-6} cm/s)
GP	2,000 to 1,000,000
GW	1,000 to 100,000
GP-SP	1,000 to 50,000
GW-SW	500 to 5,000
GM	10 to 500
SP (medium to coarse)	10,000 to 20,000
SP (fine to medium)	5,000 to 10,000
SP (very fine to fine)	500 to 5,000
SW	300 to 5,000
SP-SM	10 to 1,000
SM	10 to 500

References: [18], [26], [33], [36], [44-45]

Permeability (k_V) of Washed Embankment Drain Materials (k_V increases with grain size)

Material	k_V Range (ft/yr or x10^{-6} cm/s)
Coarse sand and gravel	150,000 to 500,000
Medium to coarse sand	50,000 to 150,000
Fine to medium sand	10,000 to 50,000

References: [18], [26], [33], [36], [45]

Anisotropy (k_H/k_V) of embankment materials (k_H/k_V increases with placement water content)

Material	k_H/k_V Range
<u>Embankment core</u> Reclamation standard placement	4 to 9
Nonstandard placement	9 to 36
Hydraulic fill	64 to 225
<u>Embankment shell</u> Reclamation standard	4 to 9
<u>Embankment drains</u> Reclamation standard	1 to 4

References: [3], [17], [21], [31], [37]

Figure 8.3.2.3.3-1. Permeability of various embankment materials.

8.3.2.4 Effect of Degree of Saturation on Permeability

The degree of saturation of a soil has an important influence on permeability, with a decrease in saturation leading to a decrease in permeability. Testing done by Lambe [46] indicates that when the degree of saturation of a soil is less than 85 percent, much of the air would be continuous throughout the soil void space, and Darcy's Law would not apply. However, when the degree of saturation exceeds 85 percent, most of the air is present as small bubbles, and Darcy's Law would be applicable. For compacted fine-grained earthfill, the degree of saturation at standard Proctor density is 75 to 85 percent. Figure 8.3.2.4-1 presents a graph based on testing by Wallace [47] that shows the effect of saturation on the permeability of certain sands.

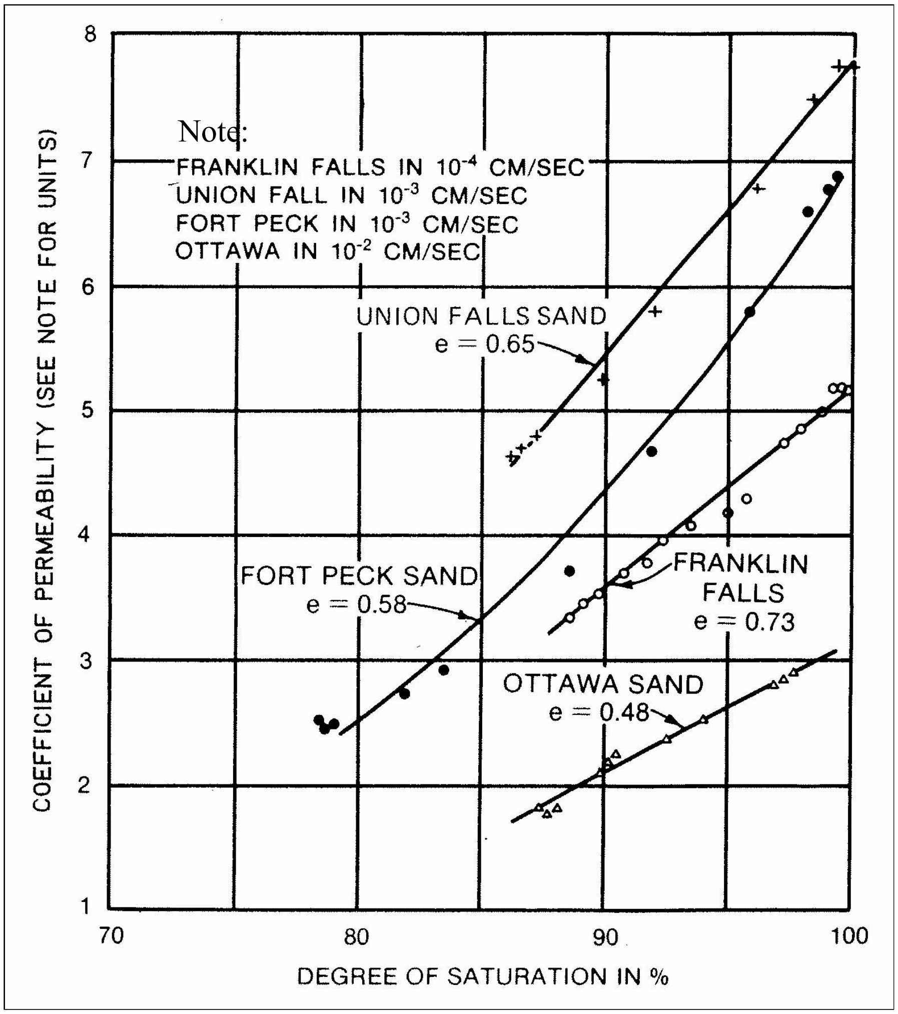


Figure 8.3.2.4-1. Permeability versus degree of saturation for various sands [47].

As described in the U.S. Army Corps of Engineers (USACE) engineering manual on seepage analysis [17], the ratio of permeability in a partially saturated versus fully saturated sand at the same void ratio is defined as:

$$k_{us}/k = 1 - m(1 - S/100) \quad \text{for } 100 \geq S \geq 80$$

Where: S = degree of saturation in percent
k_{us} = unsaturated (partially saturated) permeability
k = saturated permeability
m = constant with values between 2 (for uniform grain size) and 4 (for well graded materials)

In the previous version of Reclamation's seepage design standard, it was reported that unsaturated permeability is sometimes referred to as "relative permeability," which is simply the ratio of k_{us}/k. Furthermore, the functional relationship between relative permeability and water content was considered to be hysteretic in nature, with higher relative permeability occurring during drainage than during infiltration. However, for most practical embankment dam problems, a single-valued function was considered acceptable. The following uncited empirical relationship between saturated and unsaturated permeability (which was defined as relative permeability) was given:

$$k_r = (\theta/n)^{\varepsilon}$$

Where: k_r = relative permeability, or k_{us}/k
θ = volumetric water content of the soil
n = soil porosity
ε = empirical exponent which varies from 3 to 6, with typical values as follows:
Compacted drain materials or natural sands, use 3.5
Compacted embankment and porous rock, use 4.0
Natural soils containing fines, use 4.5

When analyzing soils that are not fully saturated (transient conditions), use of the graph in figure 8.3.2.4-1 and the above equations may provide a reasonable range of values to use for lowering the saturated permeability value. However, seepage analysis programs such as SEEP/W typically provide functions to perform this adjustment. This is usually the prudent approach since the 85 percent saturation boundary will not be known in most cases. Note also that this issue is generally not critical for our typical analyses.

8.3.2.5 Effect of Void Ratio on Permeability

It is readily apparent that the void ratio of a soil would have a significant impact on permeability because more void space will allow more water to pass through a soil. Figure 8.3.2.5-1 is a graph from Lambe and Whitman [4] that shows the relationship between permeability and void ratio for a variety of soils. The authors reported that considerable testing data suggest that for a given soil, a plot of void ratio versus log of permeability is frequently a straight line.

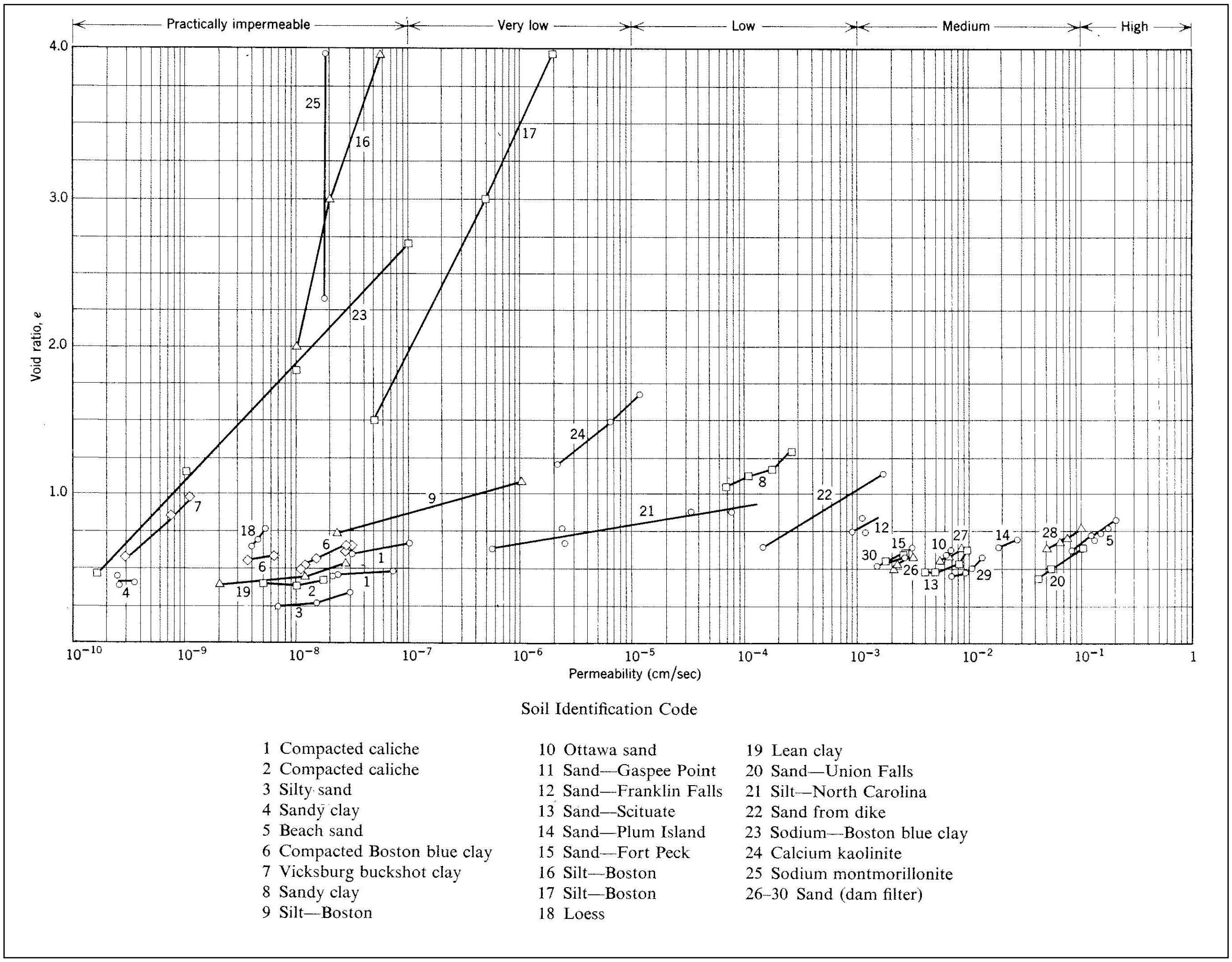


Figure 8.3.2.5-1. Permeability test data [4].

8.3.2.6 Effect of Particle Size on Permeability

Similar to void ratio, it is intuitively obvious that uniformly graded soils with larger particle sizes would contain larger void spaces and, thus, have higher permeabilities. For cohesionless soils, this is particularly important. The most well-known work in this area is from Hazen, who proposed this formula based on tests of uniform (uniformity coefficient < 5), loose, clean sands (with D_{10} ranging between 0.1 and 0.3 mm):

$$k = 100(D_{10})^2$$

Where:
- k = coefficient of permeability in cm/sec
- D_{10} = particle size in cm at which 10 percent of the material is finer (by weight)

It is important to stress that the Hazen Formula is applicable only to clean (< 5 percent silt and clay), uncemented, and uniformly graded sands and gravels. For example, this formula would be applicable to filter and drain materials. For a wider range of soils, the Moretrench Company has developed curves relating soil permeability with both D_{50} grain size and uniformity coefficient. These curves were published in reference [18] and are included as figures 8.3.2.6-1, 8.3.2.6-2, and 8.3.2.6-3.

The Soil Conservation Service published information in 1984 that also related permeability of clean sands and gravels to gradations. However, rather than develop a relationship with a given particle size, they showed permeability values for overall gradations. If readers are interested in that work, they may refer to reference [48].

For cohesive soils, there also appears to be an increase in permeability with increasing clay mineral size [17]. Figure 8.3.2.6-4 demonstrates that relationship for three common types of clays. With clays, it is also important to realize that the chemical interactions among clay minerals can be an important factor for permeability. Specifically, the ion exchange capacity of clay soils can affect permeability by a couple of orders of magnitude. However, since the permeability of a clay soil is typically quite low, this issue is generally not of concern for most embankment analyses.

8.3.2.7 Effect of Soil Fabric on Permeability

Soil fabric is typically a consideration for compacted clayey soils and refers to whether the soils are in a flocculated or dispersed configuration. Soils compacted dry of optimum tend to have particles oriented in a flocculated state (like a house of cards), which results in more open pathways for seepage and, thus, higher permeabilities. Soils compacted wet of optimum tend to have a dispersed (parallel) orientation, creating a more tortuous seepage path with smaller flow channels and lower permeabilities. (Such soils are generally less brittle as well.) Since past Reclamation practice was to construct embankment cores dry of optimum to minimize construction pore pressures, many or most of these dams may have a more flocculated than dispersed fabric and may have somewhat higher permeabilities as a result. The difference is expected to be generally around an order of magnitude and probably is of no major consequence for already low permeability clay soils.

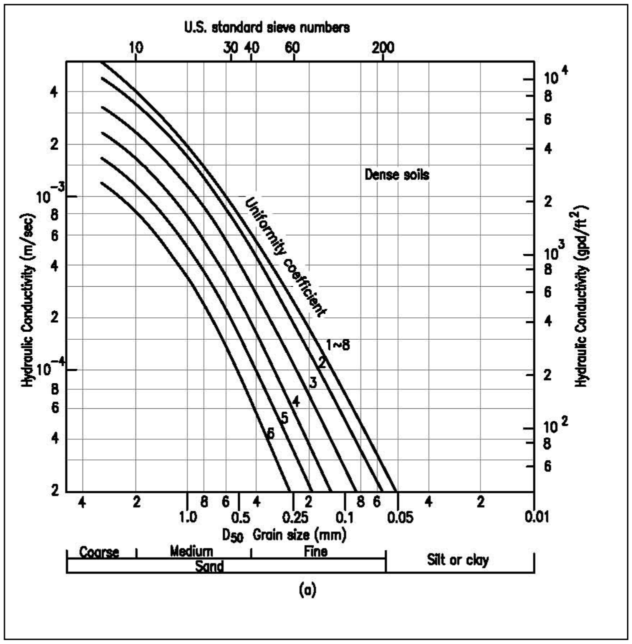


Figure 8.3.2.6-1

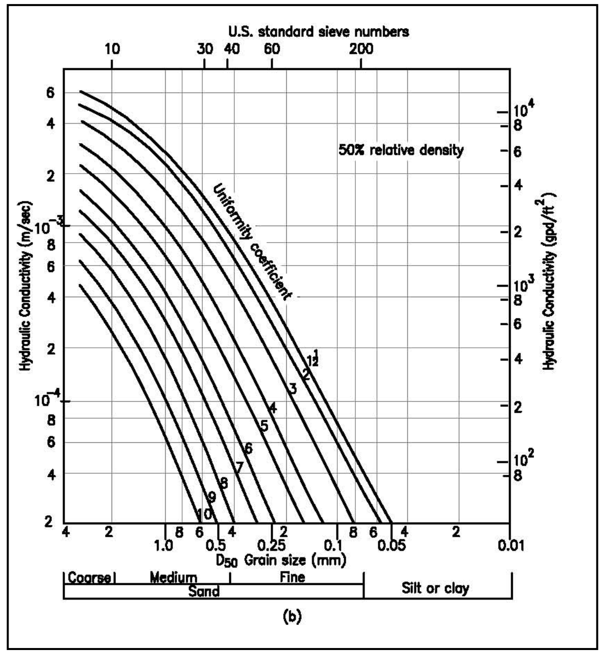


Figure 8.3.2.6-2

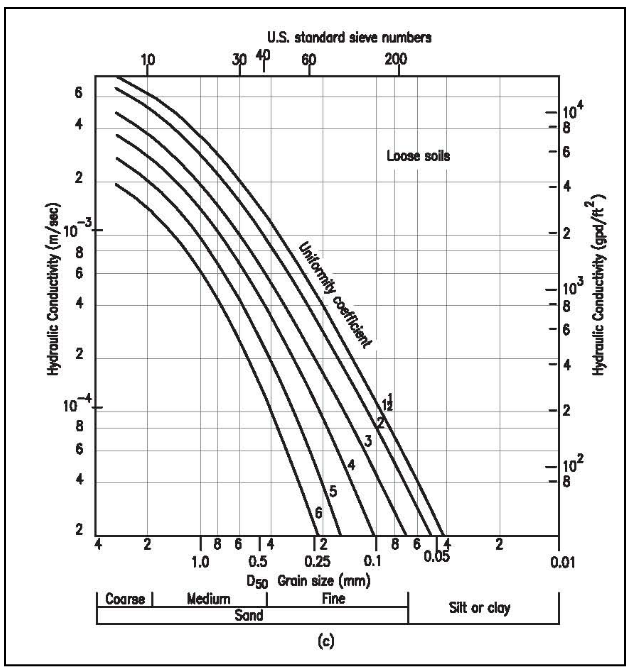


Figure 8.3.2.6-3

*** Charts courtesy of Moretrench American Corporation**

Figures 8.3.2.6-1 – 8.3.2.6-3. Permeability of noncohesive soils based on grain size, uniformity, and density [18].

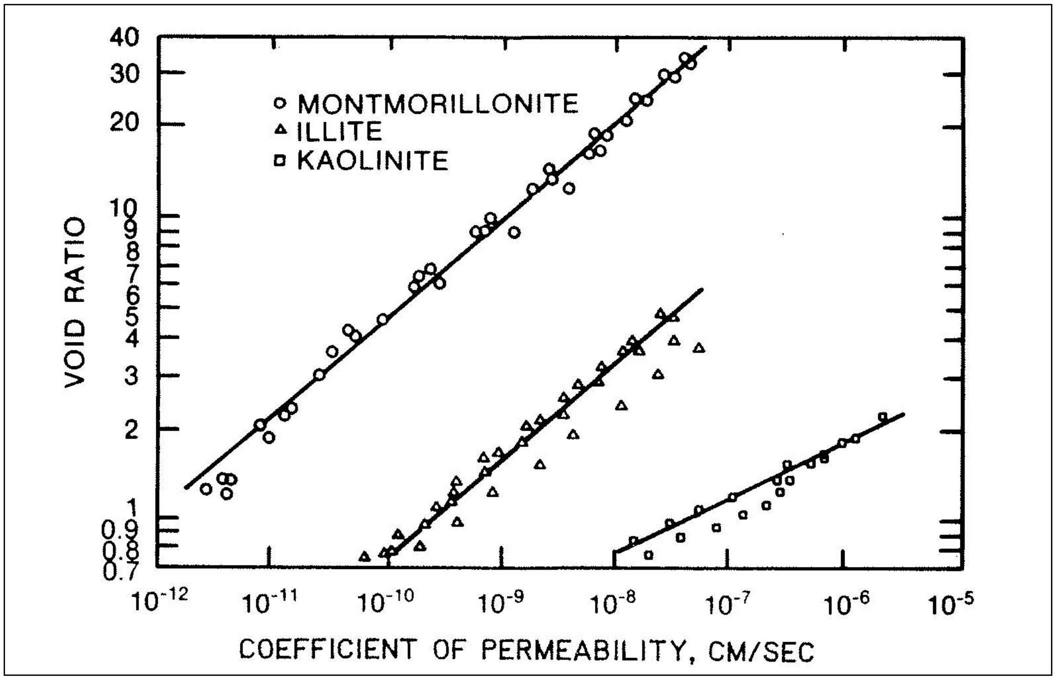


Figure 8.3.2.6-4. Increase in permeability with increasing void ratio of three different types of clays [17].

8.3.2.8 Estimating Permeability from Field Tests

In situ field tests are generally considered the most dependable means of determining the permeability of foundation materials. A number of different tests are routinely used by Reclamation, depending on the situation. These tests are large scale aquifer tests and then several types of borehole permeability tests.

Field test programs should be developed with assistance from an engineering geologist or geohydrologist using Reclamation's procedures for issuing Field Exploration Requests.

8.3.2.8.1 Aquifer Pumping Tests

Aquifer pumping tests are widely considered the most accurate (and most expensive) means of determining the horizontal permeability of a pervious stratum. These types are limited, however, to testing below the water table since the test consists of controlled pumping of a well and carefully measuring drawdown in several adjacent observation wells. Aquifer pumping tests eliminate localized effects on test results by measuring response over a finite portion of the aquifer (distance between pumped well and observations wells), rather than a very small portion, as with other methods (i.e., limited area around a borehole or the small size of a laboratory test sample). This type of test is often performed when high quality data is required for large construction dewatering designs. The specifics of this test are described in more detail in chapter IX of Reclamation's *Ground Water Manual* [15] and numerous American Society for Testing and

Materials (ASTM) procedures. Interpretation of the tests requires modeling of the process to solve for aquifer permeability, storability, and transmissivity using commercially available software.

8.3.2.8.2 Bore Hole Infiltration Tests

Bore hole infiltration tests are routinely performed for most Reclamation drilling programs. Borehole tests may either be constant head injection tests, slug tests, or recovery tests. The test can be performed in either soil or rock but is most common in rock. Water is injected into an isolated interval in a drill hole, and the amount of water injected is determined for a specific period of time. These tests are performed at preselected or continuous intervals of the bore hole both above and below the water table, and they provide good definition of the relative horizontal permeabilities of the various strata. Because they are performed at many intervals along the depth of the hole, the test time is short (normally 3 to 10 minutes). As such, numerical values of permeability may be inaccurate by a factor as large as 10, especially above the water table where saturation is incomplete. These tests are useful for determining relative permeability of various strata and for extending actual permeability data when correlated with other types of testing. The pressure applied during these tests must be predetermined and carefully controlled to prevent hydraulic spreading of existing fractures in the formation. Details, precautions, and test procedures are described in Test Designation 7310 of Reclamation's *Earth Manual*, Part 2 [19], and *Engineering Geology Field Manual* [49]. The general rule of thumb for injection pressures in soils is to not exceed ½ pound per square inch (lb/in^2) per foot of depth.

8.3.2.8.3 Bore Hole Testing in Rock

Rock water testing is normally performed in HQ3 wire line diamond drilling tooling using a packer system. Core runs in rock are generally 10 feet, and after the coring, the rods are pulled back and the packer is set and water injected. Injection pressures can be ramped up and down. These tests are more qualitative in nature with pressure gauges and flowmeters used at the surface. For rock boreholes that can stay open, water testing can be staged upwards after completion of the drill hole. For more accurate measurements, double packer tests can be used to isolate test intervals, and down hole pressure measurements can be performed. Details on the testing can be found in volume II of Reclamation's *Engineering Geology Field Manual* [49].

8.3.2.8.4 Direct Push Pneumatic Slug Testing of Soils

A pneumatic slug test system for soils below the water table has been developed for direct push testing using double tube systems (ASTM D 7242). The test employs direct push equipment using hydraulic hammering action to drive the double tube rods. Direct push testing was developed for environmental site characterization because little or no cuttings are generated in the drilling action. Direct push testing and pneumatic slug testing can be performed much more rapidly than traditional rotary drilling methods. With a double tube system, an inner sampling barrel can be used to collect continuous samples. At selected test intervals, the outside tubing is retracted, and a well screen with riser is exposed outside and below the tube. The test procedure consists of developing the well screen and then attaching a manifold with a pressure transducer that drops down into the water column. Through the manifold, the water level is either depressed or raised using pressure or vacuum through the manifold. Permeability is based on the recovery times. Multiple tests can be staged as the depth progresses. This test becomes difficult to perform for soils with permeabilities less than 10^5 cm/s.

8.3.2.8.5 Shallow Well Infiltration Tests

Shallow well infiltration, or well permeameter, tests are used to measure horizontal permeability of surficial soil deposits above the water table. The method essentially consists of measuring the rate at which water flows out of an uncased well under a constant gravity head. The hydraulic conductivity measured may not reflect the saturated permeability if the soil is to be subsequently saturated. Results can be questionable for finer grained soils with permeabilities less than 10^{-5} cm/s. The average permeability for the portion of the well that is filled with water is determined after the infiltration rate has stabilized for a constant head. The specifics of this test are described in more detail in chapter III, Part B of Reclamation's *Drainage Manual* [20], as well as in Test Designations 7300 and 7305 of Reclamation's *Earth Manual*, Part 2 [19].

8.3.2.8.6 Ring Permeameter Tests

Ring permeameter tests are designed to measure the vertical permeability of surficial deposits above the water table. Tests can be performed at selected levels during excavation of a test pit to determine the variations of permeability with depth. The hydraulic conductivity measured may not reflect the saturated permeability if the soil is to be subsequently saturated. Results can be questionable for finer grained soils with permeabilities less than 10^{-5} cm/s. The specifics of this test are described in more detail in chapter III, Part B of Reclamation's *Drainage Manual* [20]. Sealed double ring infiltrometer tests (ASTM D 5093) are designed for surface testing soils with permeabilities less than 10^{-5} cm/s.

8.3.2.9 Estimating Permeability from Laboratory Tests

There are several laboratory tests that can be performed to estimate permeability of soil and rock materials. In Reclamation studies, these tests tend to be most common on compacted embankment soils because remolded samples can be prepared as opposed to attempting to retrieve, transport, and test undisturbed samples. Laboratory permeability tests are viewed by some to be most useful for testing on fine-grained embankment soils because a large range of gradients can be imposed, and the higher gradients make test times shorter than for field tests. A significant weakness of laboratory tests is that while they can be accurate for the small sample tested, the sample itself may not be representative of the large volume of in situ material. Laboratory test results on undisturbed samples of soil and rock tend to show lower permeabilities (in some cases by orders of magnitude) than field tests of the same material. Large scale features that can control permeability, such as cracks in soil or joints in rock, are unlikely to be adequately captured by small laboratory samples. Other concerns include difficulties in orientation of samples to ensure whether a horizontal or vertical permeability is being measured, and the tendency for air bubbles to occur during some laboratory tests (i.e., in tests involving rigid-wall permeameters), which can lead to significant underestimation of permeability. For these reasons, laboratory tests are typically viewed as less reliable than field tests for estimating permeability. If laboratory testing is determined necessary, a program should be developed with engineers form the Materials Engineering and Research Laboratory.

The typical laboratory permeability tests are either constant head or falling head tests. In essence, these are controlled tests in which the cross-sectional area, the hydraulic gradient, and the quantity of flow can be measured. Permeability can then be calculated from these values using Darcy's Law.

Following are brief descriptions of laboratory tests Reclamation has typically used.

8.3.2.9.1 Reclamation's Permeability Settlement Constant Head Test

Reclamation routinely ran constant head permeability and settlement tests in rigid wall permeameters during construction of dams from the 1950s to 1970s. These tests were run for design but also as record tests, typically for every 30,000 cubic yards (yd^3) of compacted soils during construction. Construction reports (L-29 reports) report this data during construction. This data may be valuable for seepage evaluations of existing structures. The permeameters were designed to simulate application of overlying fill by applying loads to the top plate of the apparatus. Therefore, there is an added benefit of measuring the settlement of compacted fill upon saturation.

For minus No. 4 sieve size soil, a 3-inch-thick soil specimen (either remolded or undisturbed) is placed in an 8-inch inside diameter, rigid-wall permeability cylinder. For minus 3-inch particle size soil, a 9-inch-thick soil specimen is compacted in a 19-inch-diameter rigid wall apparatus. Water from a constant head tank is passed through the specimen (after it is thoroughly wetted) under selected hydraulic gradients, and flow rates are measured once they are stabilized. Deformations are also measured. Once all of the pressure gradients of interest have been tested, the confining pressure can be increased and testing continued. Details, precautions, and test procedures are described in Test Designations USBR 5600 and 5605 of Reclamation's *Earth Manual*, Part 2 [19].

Since the saturation of the soils with permeability less than 10^{-5} cm/s cannot be ensured by just gravity percolation without back pressuring, the results for soils with a lower permeability are questionable.

8.3.2.9.2 Constant Head Test

The standard constant head test for granular soils in a rigid cell (ASTM D 2434) can be performed on soils with permeabilities greater than 10^{-5} cm/s. This simple test is only performed on recompacted soil specimens. A vacuum pressure can be used to draw water from the bottom to the top of the specimen to saturate the specimen. Constant head flow can be applied with different gradients. This test could be useful for checking hydraulic conductivity of free draining zones such as filters and transition zones in embankments.

8.3.2.9.3 Back Pressure Test with Flexible Membrane

This test is different from the preceding tests in that a specimen is not constrained within a rigid-wall cylinder but, rather, within a membrane by application of cell confining pressure, similar to a triaxial test. After a high degree of saturation is achieved by back pressure, water is allowed to flow through the specimen. The back pressure test allows assessment of permeability (as low as 10^{-9} cm/s) at various combinations of effective confining pressure and hydraulic gradients. The test is not applicable to cemented soils or rocks which cannot be effectively back pressured with water, including cemented slurry wall materials. The standard test is a falling head, rising tailwater test using small diameter manometer tubes. For extremely low permeability soils, a variation of this test uses fluid pumps, and the gradient is measured using differential pressure transducers on the specimen end plates. This test can be performed on recompacted or intact soil or rock specimens. Tests can be performed on specimens oriented in different directions to evaluate anisotropy. Generally, this procedure results in a more reliable and accurate coefficient of permeability, particularly for fine-grained soils, that is obtained from falling head or constant head tests in rigid-wall permeameters where provisions are not available to remove air trapped in the soil voids. Details, precautions, and test procedures are described in Test Designation 5610 of Reclamation's *Earth Manual*, Part 2 [19] and ASTM D 5084.

8.3.2.9.4 Constant Head Test for Primary Permeability of Porous Rock

This test is performed on cylindrically shaped rock specimens under vacuum and constant head. Porous rock, rock with defects, or soil-cement lift lines have been tested. The fluid flows radially inward through the specimen to a ½-inch-diameter hole drilled lengthwise through the vertical axis of the specimen. After the specimen is placed in a chamber, a vacuum is applied to the specimen through the centrally drilled hole, and the waterflow is measured during a given time interval. The primary permeability of the rock can then be calculated. Details, precautions, and test procedures are described in Test Designation 5615 of Reclamation's *Earth Manual*, Part 2 [19].

8.3.2.9.5 Falling Head Test for Nongravelly Soils

This test is simply the traditional one-dimensional (1D) consolidation test, of which permeability of the specimen can be measured using a head tube connected to the base of the specimen. It is typically limited to soils with permeabilities greater than 10^{-5} cm/s, depending on the size of the consolidometer. Instead of a constant head, the water level supply for this test is allowed to fall but is carefully measured. For clay soils, it is possible to measure permeability by evaluating the time compression curves at different loads. The rate at which consolidation occurs is a function of the permeability. Details, precautions, and test procedures are described in Test Designation 5700 of Reclamation's *Earth Manual*, Part 2 [19].

8.3.2.10 Pertinent Construction/Investigation Data for Estimating Permeability

For existing dams, it is important to search through project records to locate any past data, studies, or details that would aid in the understanding of permeabilities at the site. Such information may include:

- Field or laboratory permeability testing (that may have been performed before or during construction; Reclamation L-29 construction reports often contain test results)

- Material property data (particularly gradations), which can be used to estimate permeabilities from published data (for Reclamation features, data exists in our laboratory database of reports)

- Geology reports and logs of drilling, which may contain information on borehole water tests, fracture density and aperture sizes, foundation soil characteristics and stratigraphy, presence of soluble rock units, etc.

- Grouting records containing information on water test results and grout takes in rock units

- Construction photos, which can provide information on the types of materials used in construction or encountered in the foundation

For new dams, similar type data may be available for other dams or projects in the nearby area, including other civil works investigation or construction, oil and gas drilling explorations or operations, water well drilling records, regional geology studies, etc.

8.3.3. Piezometer and Seepage Readings

For existing dams, instrumentation data regarding piezometric levels and seepage flow rates can provide useful insights for seepage evaluations and analyses.

8.3.3.1 Piezometer Data

Following are key uses of piezometric data in seepage evaluations and analyses, in no particular order of importance or usefulness.

8.3.3.1.1 Determination of Phreatic Surface

An obvious use of piezometers is to help determine the phreatic surface within the embankment. The phreatic surface is defined as the surface in the embankment and/or foundation along which the pore pressure is equal to atmospheric pressure. In other words, the phreatic surface represents the zero pressure contour line in a seepage analysis and the uppermost limit of any seepage flow path.

An important distinction to make is that the phreatic surface is **not** equivalent to the piezometric line. The piezometric line used in limit equilibrium stability analyses (such as with SLOPE/W) is an assumed line reflecting the pore pressures within the embankment. This line is used to estimate pore pressures at the base of each slice in the stability analysis by simply using the vertical distance between the piezometric line and the base of the slice and multiplying by the unit weight of water. The phreatic surface is only equivalent to the piezometric line used in stability analyses for conditions of purely horizontal flow or under hydrostatic conditions. For embankments, the flow lines are usually dipping downward; therefore, the equipotential contours are not vertical, as shown on figure 8.3.3.1.1-1. The phreatic surface shown in the figure is thus higher than would be assumed by measuring water levels in vertical drill holes due to considerable head loss. In other situations without much head loss, such as through a higher permeability layer, piezometric levels can be higher than the phreatic surface. Thus, when using piezometers to calibrate seepage models with piezometer data, it is important to understand the distinction between the phreatic surface and measured piezometric levels.

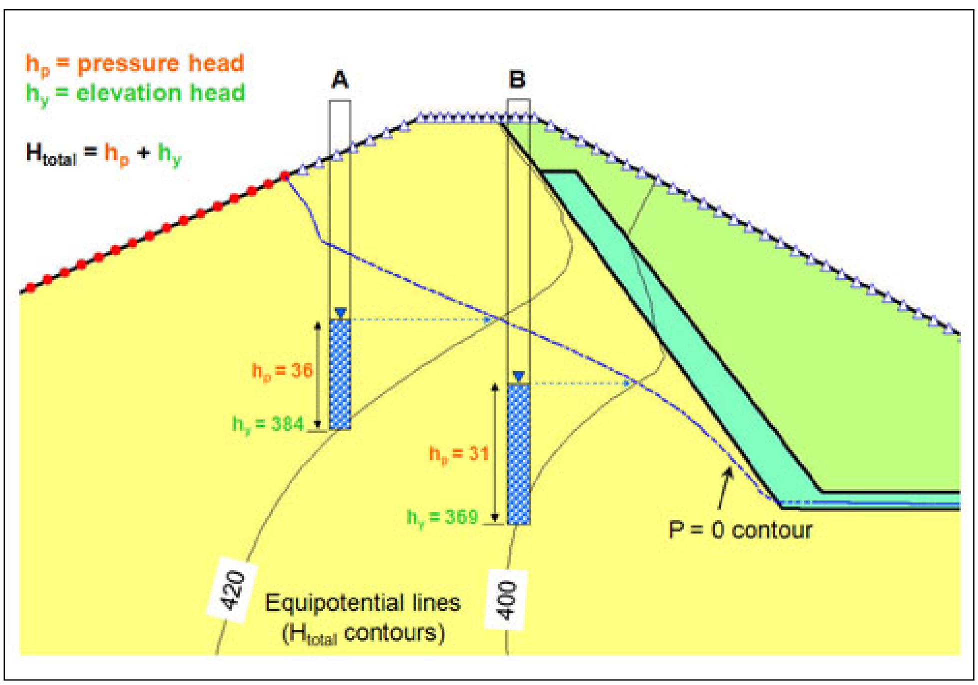


Figure 8.3.3.1.1-1. Piezometric water levels.

8.3.3.1.2 Location of Seepage Pathways

An array of piezometers within an embankment or foundation can provide an indication of seepage pathways. With multiple piezometers, it may be possible to plot contours of piezometric head, which may provide a meaningful way of identifying which areas of the dam or foundation have greater permeability.

8.3.3.1.3 Development/Progression of Seepage Failure Modes

Changes in piezometric levels with time (corrected for changing reservoir levels) are an obvious indication of changes in the seepage behavior within a dam or foundation. Unexplained increases or decreases (fluctuations) in pressures could indicate an episodic internal erosion process, where changing pressures may be a result of periodic or intermittent opening up and plugging (self-healing) along an internal erosion pathway. Any change in piezometric levels from what has been observed in the past deserves close evaluation.

> **Note:** For any evaluation involving piezometric behavior, it is important to recognize that the critical seepage path in any embankment or foundation may not occur in an obvious location, and the probability is low that a piezometer is located in the precise area to measure important seepage and internal erosion behavior in the "weak link" or critical portion of the seepage path. This does not diminish the overall value of piezometers but does mean that satisfactory piezometric behavior should not be construed as an indicator that seepage or internal erosion elsewhere in the dam is not possible/occurring.

8.3.3.1.4 Estimation of Hydraulic Gradients

Multiple piezometers at the toe of an embankment with different vertical influence zones at different elevations can enable the estimation of vertical exit gradients and provide data for evaluating the stability against uplift. In these cases, it is important to closely look at the defined influence zones to ensure that pressure readings are isolated in a given pervious stratum or impervious layer. Influence zones that span multiple soil layers will likely not provide sufficient data to make accurate estimates of vertical gradients. Piezometer readings tend to reflect the most pervious layer within an influence zone. With only a single piezometer at the toe, assumptions must be made about piezometric levels in higher or lower portions of the foundation, and the accuracy of the gradient estimates will be significantly more suspect.

Two or more piezometers located along horizontal layers in an embankment or foundation can provide data with which to estimate horizontal gradients along a seepage path. Since seepage paths are likely to meander in many natural (or even compacted) materials, there will always be uncertainty about whether the gradients reflect the true condition along a potential seepage pathway.

8.3.3.1.5 Determination of Pore Pressure Effects on Embankment Stability

Piezometers in the embankment and foundation provide key information for determining whether detrimental high pore pressures exist that may lead to instability of the embankment. Should increased piezometric levels occur under flood loadings or other high reservoir situations, it is essential to record and document the reservoir level and associated piezometer readings to identify any increased levels in internal pore pressures. To estimate the potential change in pore pressures without actual higher reservoir level data, piezometric trends as displayed in "scatter plots" (piezometric level versus reservoir level) can be extrapolated to expected future reservoir levels.

8.3.3.1.6 Model Calibration

An array of piezometers can provide very useful data for calibration of seepage models. As mentioned earlier under the discussion of the phreatic surface, it is essential to understand the location of the piezometers with respect to expected flow lines so that an accurate calibration can be made. In addition, piezometric readings can be matched to pressures developed in the models, which provides a more rigorous calibration than with just the phreatic surface. An analyst should always be wary of making unrealistic assumptions just to force a calibration with existing data. Piezometers can be located either in or out of a key flow path and, thus, may not be representative of overall conditions. As the number of available piezometers increases, so do the chances of improving the calibration of the model to actual site conditions. It is important to remember that in seepage modeling, there can be multiple valid solutions for any given problem; engineering judgment is required throughout the calibration process.

8.3.3.2 Seepage Data

For new dams, the presence of existing seeps or springs should be carefully noted, so that subsequent seepage performance after reservoir impoundment can be related to preexisting behavior if necessary. For existing dams, seepage data can be a valuable source of data or observations for evaluating seepage issues or developing seepage analysis models. Following are key uses of seepage data in seepage evaluations and analyses, in no particular order of importance or usefulness.

8.3.3.2.1 Location of Seepage Pathways

The appearance of seepage within certain areas of an embankment or foundation is an obvious indication of seepage pathways. The relative amount of flows in seepage areas suggests the significance of seepage pathways. The location of the surface seeps can provide insights into the potential risks of an internal erosion failure. For example, seepage emanating near the downstream end of an outlet works conduit, or along the side of a spillway wall, could indicate the potential for internal erosion at the embankment/structure interface. Keep in mind that important seepage exit locations may be hidden beneath a zone of rockfill or in an unlined stilling pool.

8.3.3.2.2 Signs of Particle Transport

Properly designed seepage weirs (or similar flow measurement devices) have an appropriately sized pool behind them to "still" the flows and allow soil particles to settle out of the flowing seepage. Thus, seepage measurement sites are well suited to monitor for signs of soil particle transport, which is generally considered direct evidence that a potential internal erosion event may be in progress. A less preferred alternative to a stilling pool is a turbidity monitoring unit, which uses changes in the optical clarity of water to indicate the presence of suspended soil fines in a seepage flow. While seemingly sound in concept, in practice, turbidity monitoring units have been found to be ineffective in providing useful information regarding sediment transport by seepage flows. When a seep is not quantitatively measured for flow, perhaps due to a difficult location or very low flow rates, visual evidence of sediment transport (e.g., material transport along flow paths) is nonetheless very important. However, all seeps should have their flows quantitatively monitored when at all possible.

8.3.3.2.3 Development/Progression of Seepage Failure Modes

Changes in seepage flows with time (corrected for changing reservoir levels) are an obvious indication of changes in the seepage behavior within a dam or foundation. Unexplained increases or decreases (fluctuations) in flows could indicate an episodic internal erosion process, where changing flows may be a result of periodic or intermittent opening up and plugging (self-healing) along an internal erosion pathway. Any change in seepage flows from what has been observed in the past deserves close evaluation.

8.3.3.2.4 Model Calibration

Measurement of flow rates can be used to calibrate seepage models. In some cases, total head at a seepage location can be measured and related to the seepage model. This can be done by stacking up a sandbag ring around a seepage exit point until the flow ceases, and then measuring the depth of water. In addition, toe drain flows can be used to calibrate models. A caution with the use of any measured flows is that the flows may come from many sources (have a three-dimensional [3-D] component) which may not be able to be properly considered in a two-dimensional (2-D) model.

8.3.4 Geophysics

The field of geophysical testing continues to improve techniques for locating seepage patterns within existing dams and their foundations [50]. Self-potential, electrical resistivity imaging and electromagnetic surveys have had some success with locating preferred seepage pathways in embankments. Often, such surveys are conducted at both low and high pools to better detect seepage anomalies.

Geophysical methods can also play a role in locating anomalies that may result from seepage flows. Side scan sonar surveys in reservoirs can provide surprisingly clear images of potential sinkholes or seepage entrance points. Ground penetrating radar can be useful in detecting voids or anomalies within embankments or behind and below concrete linings or conduits. As with most geophysics techniques, confirmation explorations are typically also conducted to better correlate and verify the findings from geophysical testing.

8.3.5 Dye Tests

Dye tests are frequently used in investigations of existing seeps. Dye is introduced into a seepage entrance point, and downstream areas are constantly monitored to determine where and when traces of the dye appear. Such tests can provide evidence of the location and continuity of seepage paths. By measuring travel times of the dye, estimates of velocities along the seepage pathway can be made, which can be further converted to permeabilities by making assumptions on the area of the pathway.

A number of dye types exist. The most common is a visual dye such as rhodamine red, which can be detected by the unaided eye. This dye will have to be present in high concentrations in order to be seen, a condition that may not be practical for long seepage paths or where flow rates are high. Also available are fluorescent dyes, which can be detected at smaller concentrations with the use of a ultraviolet sensors or flowmeters. It should be recognized that some dyes contain organic constituents and will not work in reservoirs that contain common types of bacteria.

Dyes can also be used during underwater examinations. Experience has shown that distribution of dye over a suspect area, such as the upstream side of the dam, can help divers visually identify water entering the dam or foundation at a flow rate than cannot be felt.

8.3.6 Chemical Analysis of Water, Soil, and Rock

When dissolution is a potential issue at a dam, chemical analysis is recommended. For a new dam, analysis of the water source for the reservoir, the soils, and, particularly, the bedrock formation materials should be tested for chemical composition. For soluble materials, the composition and pH of the water can play an important role in whether further dissolution is possible and at what rate it may occur. In general, this relates to whether the reservoir water is undersaturated or oversaturated with the soluble minerals in the foundation.

For an existing dam where dissolution is a possible concern, a program of periodic water quality tests can indicate whether dissolution is occurring or changing. Samples of seepage water from piezometers believed to be in the flow path and from downstream surface seep locations should be taken at the same time as the reservoir water is sampled. All samples can then be chemically analyzed for the percent of various minerals or compounds present. By comparing the downstream seepage waters with the reservoir water, it is possible to detect whether dissolution is occurring and what type of formation is being dissolved. Furthermore, by looking at the relative amounts of dissolved minerals or compounds in the seepage water and knowing the flow rate, a rough estimate can be made of the amount of foundation material being removed.

Water chemistry testing can also be used to determine seepage flow paths through, under, and around a dam. In this technique, water samples are taken in the reservoir, piezometers, and surface seeps, typically during high reservoir. Water chemistry tests are performed on each sample, and a diagram (known as a Stiff diagram) is made of the major chemical constituents. These diagrams are then compared, and consideration is given to where ions can be accumulated from the earthfill or foundation soils to ascertain flow path locations. Reclamation performed this type of evaluation at McKay Dam.

For more details on water quality testing, readers should consult Reclamation's *Seepage Chemistry Manual* [51].

8.3.7 Temperature Data/Analysis

Although not widely used in Reclamation to date, temperature data or thermal monitoring can be used in seepage evaluations. This technology is discussed in Section 8.6.2.5, "Thermal Monitoring."

8.3.8 Exploring for Cracks and Voids by Trenching

Since many embankment dams are cracked in the upper portion of the dam due to settlement, desiccation, or other reasons, trenching is often used to find cracks and determine their depth, width, and orientation. Careful attention should be devoted to avoid compromising the integrity of the dam while trenching. Target locations in the dam crest where cracks may be more prevalent include near the ends of the dam, over bedrock topographic anomalies, or above conduits. Probing may aid in finding the extent of cracks and voids or help follow a stoping condition or sinkhole to its source. It is important to document the trenching with detailed mapping of the trench walls and floor.

8.3.9 Erodible Soils

The potential for internal erosion failures at a dam is highly influenced by the erodibility of the embankment and foundation soils. The erodibility of soils varies over many orders of magnitude. Parameters that affect erodibility of soil are in situ stress, soil fines and clay size content, plasticity, dispersivity, compaction water content, density, degree of saturation, clay mineralogy, and the possible presence of cementation. In general, the most dominant factor for soils is the plasticity index. For compacted soils in dams, there have been numerous studies on the effects of degree of compaction and molding water content on a wide variety of soils [52,53]. In general, research shows compaction at optimum conditions creates a soil structure most resistant to erosion. Higher compaction effort increases erosion resistance; however, soil properties including texture and plasticity influence erosion resistance as much or more than compaction factors.

Several tests have been proposed for evaluation of soil erodibilty, but the most common are the Hole Erosion Test (HET) and Jet Erosion Test (JET). The test is like the pinhole test but larger and typically the size of a compaction mold. The HET was developed by Wan and Fell for simplified risk evaluations of potential for piping and internal erosion. Wan and Fell proposed six indices called the Hole Erosion Index (I_{het}), as shown on figure 8.3.9-1 [52]. Reclamation has the capability to perform the HET, but some soils cannot be easily tested. Group numbers 1 and 2 generally fall apart upon saturation, and there could be insufficient head to test group 6 and initiate erosion.

The JET test is often used when true erosion properties of the soil are needed to evaluate the potential for initiation of erosion and erosion rate. It can also be used for general guidance to estimate dam breach erosion rates and the time to failure due to internal erosion. The test consists of concentrating a jet of water on an imersed sample in a container box [53]. A depth gauge is used to measure the amount of erosion. Reclamation has the capability to perform JET testing and is recommended over HET testing when accurate erodibilty data are needed. The test can be performed in the field or in the laboratory on intact samples as well as re-compacted soil samples. In general, this test is considered to provide better estimates of the erosion rates of soils and the critical tractive forces needed to iniate erosion.

Group number	Erosion rate index	Description
1	<2	Extremely rapid
2	2–3	Very rapid
3	3–4	Moderately rapid
4	4–5	Moderately slow
5	5–6	Very slow
6	>6	Extremely slow

Figure 8.3.9-1. Qualitative terms for representative erosion rate index (Wan and Fell, 2004) [52].

A special case of erodible soils includes dispersive clays. Dispersive clays are highly erodible and are so named because they readily disperse, or go into suspension, in the presence of water. The high degree of erodibility tends to be a function of the chemistry and mineralogy of the soils. Typical index tests such as gradation or Atterberg limits are not helpful in identifying dispersive soils. Instead, Reclamation typically uses three standardized tests to indicate whether a given soil is dispersive, and to what extent. These tests are the Crumb Test, the Double Hydrometer Test, and the Pinhole Test. Details of these test procedures are described in Test Designations 5400, 5405, and 5410 of Reclamation's *Earth Manual,* Part 2 [19]. The pinhole test is considered the preferred test for evaluating dispersive clays, and the other tests give more qualitative results.

If dispersive soils are suspected at a dam, these tests are recommended because an internal erosion failure is more likely and would occur much more rapidly in these types of soils. Dispersive soils exist in many parts of the United States and around the world. Potentially suspect areas may be identified at locations where severe erosion is noted in fine-grained exposed road cuts or similar exposed slopes.

8.4 Seepage Analysis Principles and Procedures

8.4.1 General

This section on seepage analysis deals primarily with the theory and methodology involved in computational methods of estimating embankment dam and foundation response to seepage. A brief review of theory is followed by a discussion of the different analysis methods, as well as a discussion of applicability of the analyses to geotechnical evaluation of embankment dams.

8.4.2 Darcy's Law

The start of logical analysis of seepage has been attributed to the publishing of Darcy's Law in 1856. Henri Darcy was a French engineer who conducted a series of experiments on vertical flow through small specimens of sand. His experiments demonstrated that laminar seepage flow was related to the cross-sectional area of the sand specimens and the difference in hydraulic gradient imposed on the sand. Two of the common forms of Darcy's Law are:

$$q = kiA$$

and

$$q = VA$$

Where: q = rate of discharge
k = coefficient of permeability

i = hydraulic gradient
A = cross-sectional area of flow
V = apparent velocity of flow

Note that v is the apparent velocity; it is not the actual velocity, which would be greater. The law is reasonable for most soils, but flow through coarse gravels and openings in rock may become turbulent and violate Darcy's assumptions. For estimates of velocities in turbulent conditions, Cedergren [3] serves as a good reference. Furthermore, in strictest application, Darcy's Law is only applicable to flow through saturated materials. The remainder of this section deals mostly with the analysis of assumed laminar flow through saturated soils under steady-state conditions.

8.4.3 Laplace Equation

Seepage analyses depend on a model or equation that describes the phenomena of seepage once boundary conditions and material properties are supplied. The flow of water through a porous medium like soil can be represented by the Laplace equation, which forms the mathematical basis for most models or methods of seepage analysis.

In mathematics, the Laplace equation is a partial differential equation important in many fields of science (including electromagnetism, astronomy, and fluid dynamics) because it can describe the behavior of electric, gravitational, and fluid potentials. As it applies to seepage flow, the fundamental theory is that the quantity of water entering an element must be equal to the amount leaving the element. For seepage analyses, the Laplace equation (in three dimensions) takes the form:

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0$$

Where: The terms u, v, and w are discharge velocity components in the directions x, y, and z

Using Darcy's Law, the equation can be put in terms of gradients and permeabilities, as long as the following assumptions are made:

1. The soil is homogeneous.
2. The voids are completely filled with water (i.e., saturated).
3. No consolidation or expansion of the soil takes place.
4. Both soil and water are incompressible.
5. Flow is laminar, and Darcy's Law is valid.

The resulting forms of the Laplace equation govern the steady flow of water in a porous media:

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0 \qquad \text{for 2-D flow}$$

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad \text{for 3-D flow}$$

Where: φ = k × h
k = permeability (homogeneous and isotropic)
h = total head
x, y, z = coordinate direction

For detailed discussion and theory of the Laplace equation for seepage flow, refer to Terzaghi and Peck [2] and Cedergren [3].

8.4.4 Analysis Methods

There are several means of analyzing seepage problems, most of which incorporate Darcy's Law and involve solving the Laplace equation discussed above. Analysis methods range from simple graphical approaches to detailed numerical analyses. The more typical methods are described below.

8.4.4.1 Graphical Methods

Some seepage problems can be evaluated through the use of graphs and charts available from published literature. These simplified methods based on saturated flow theory and highly idealized conditions may be appropriate for preliminary evaluations of seepage issues. Various charts used for these purposes can be found in appendix B and deal primarily with graphical solutions and equations that estimate the effectiveness of design features such as cutoff trenches or walls, upstream blankets, and relief wells.

A widely used graphical method to estimate the location of the phreatic surface within an embankment was developed by Casagrande [21]. Figures 8.4.4.1-1 through 8.4.4.1-3 illustrate the graphical method of developing a phreatic surface using the Casagrande construction.

Perhaps the most widely known graphical technique for evaluating seepage is the use of flow nets. This technique is discussed separately in section 8.4.5.

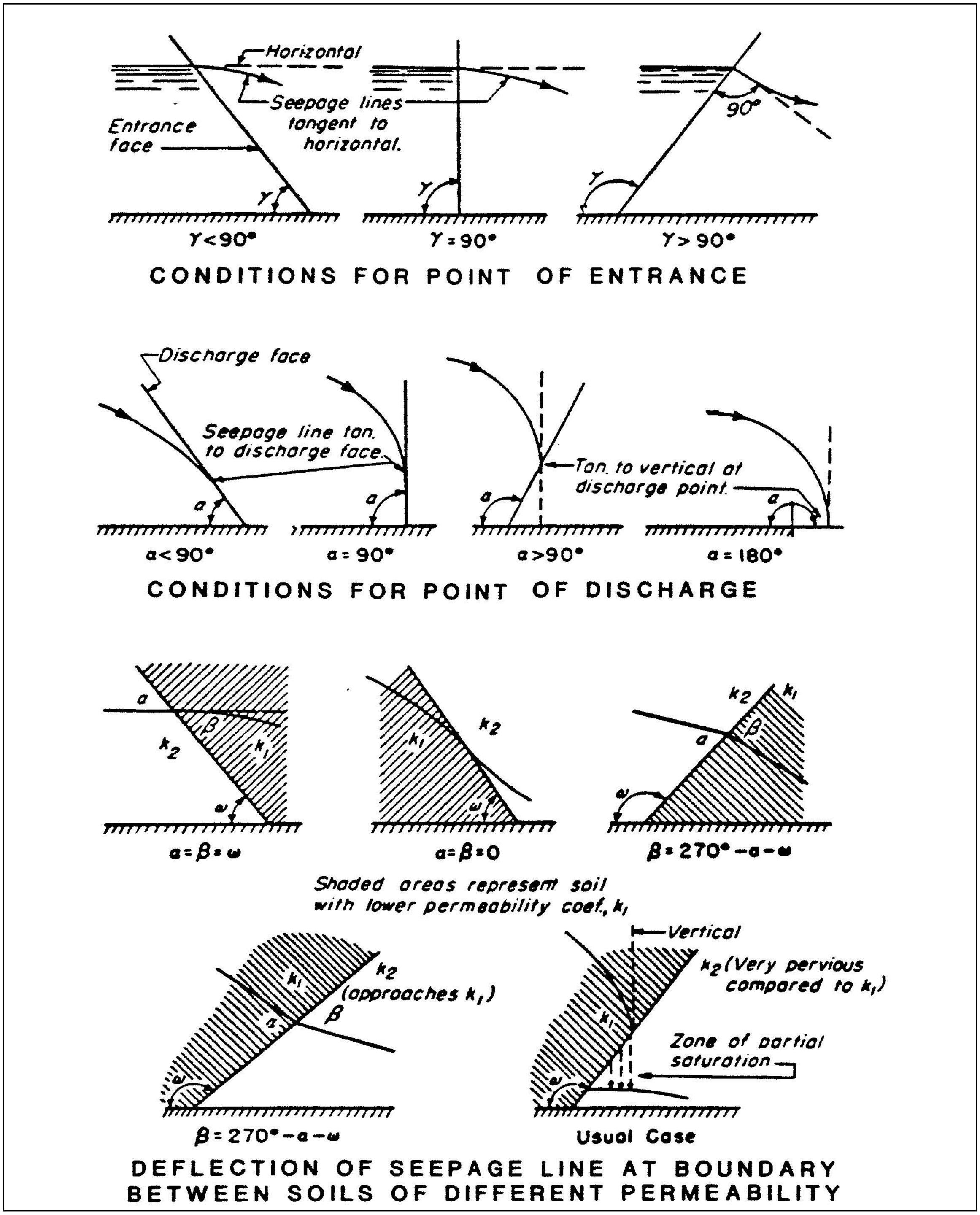
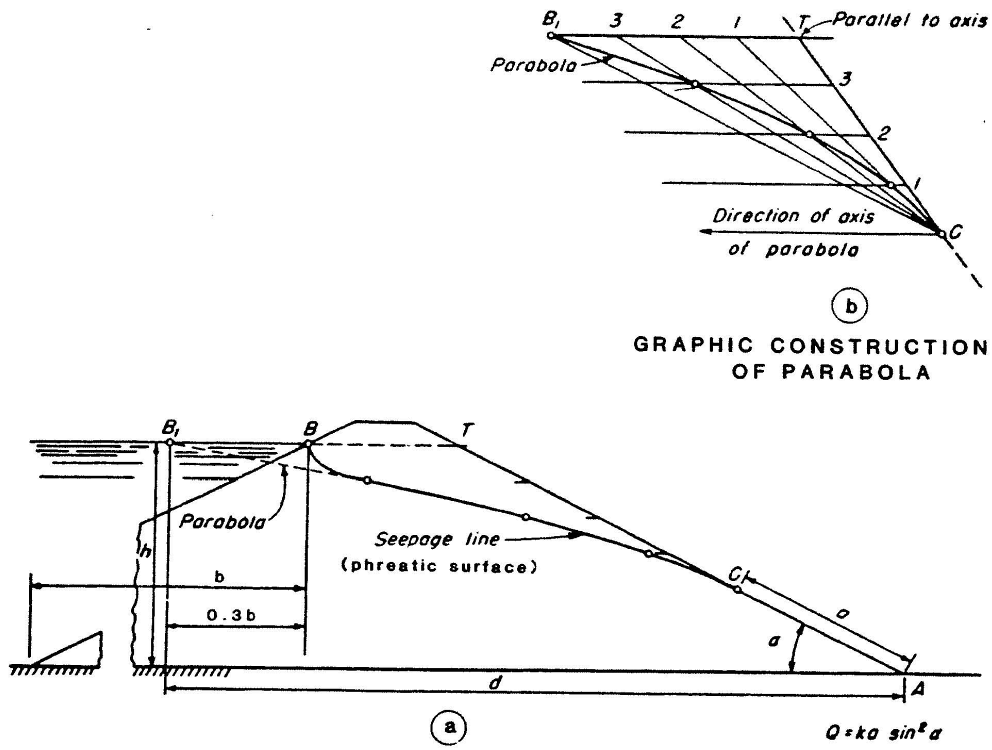


Figure 8.4.4.1-1. Entrance, discharge, and transfer conditions of seepage line [21].



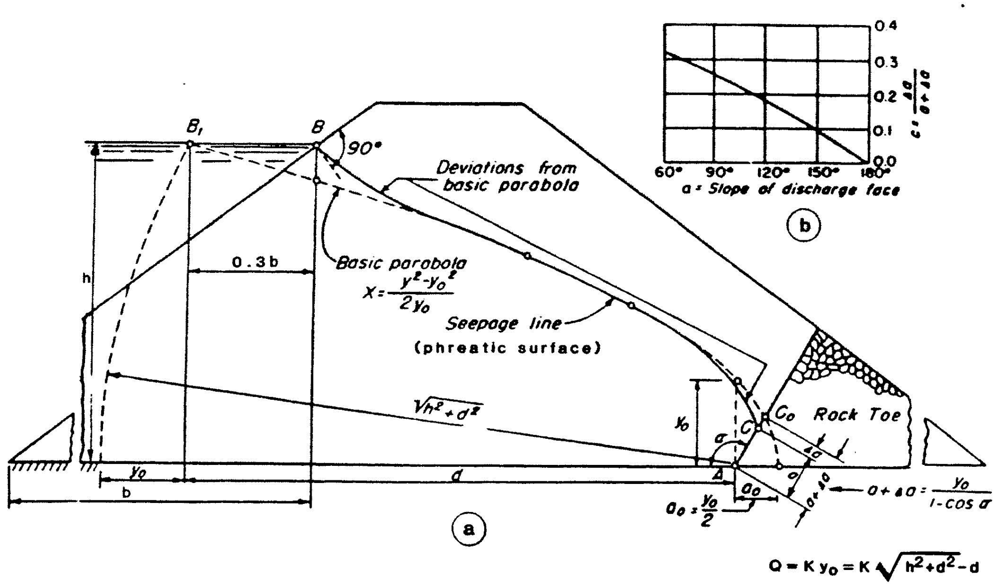
PROCEDURE FOR LOCATING SEEPAGE LINE:

1. Transform the section as in FIGURE B-2, if necessary
2. Locate point B_1, $\overline{BB_1} = 0.3b$
3. Locate discharge point C from equation

   $a = S_0 - \sqrt{S_0^2 - \frac{h^2}{\sin^2 \alpha}}$, where S_0 is length of seepage line $\overline{B_1 \widehat{C} A}$,

   $S_0 = \sqrt{h^2 + d^2}$
4. Plot parabola by graphic method as shown in (b)
5. Complete seepage line by sketching a transition curve connecting point "B" with parabola
6. Retransform the section as in FIGURE B-2

Figure 8.4.4.1-2. Determination of seepage line for homogeneous section on impervious foundation α<60° [21].



PROCEDURE FOR LOCATING SEEPAGE LINE:

1. Transform the section as in FIGURE B-2, if necessary
2. Determine point B_1, $\overline{BB_1} = 0.3b$
3. Determine y_o graphically as shown or from equation $y_o = \sqrt{h^2 + d^2} - d$
4. Plot basic parabola from equation ...... $X = \frac{y^2 - y_o^2}{2y_o}$ or determine graphically as in FIGURE C-2 (b)
5. Determine distance $a + \Delta a$ and compute Δa from chart (b)
6. Locate point C and sketch by eye the transition curve from point C to basic parabola
7. Complete seepage line by sketching the transition curve connecting point B with parabola
8. Retransform the section as in FIGURE B-2

Figure 8.4.4.1-3. Determination of seepage line for homogeneous sections on impervious foundations α>60° [21].

8.4.4.2 Physical Models

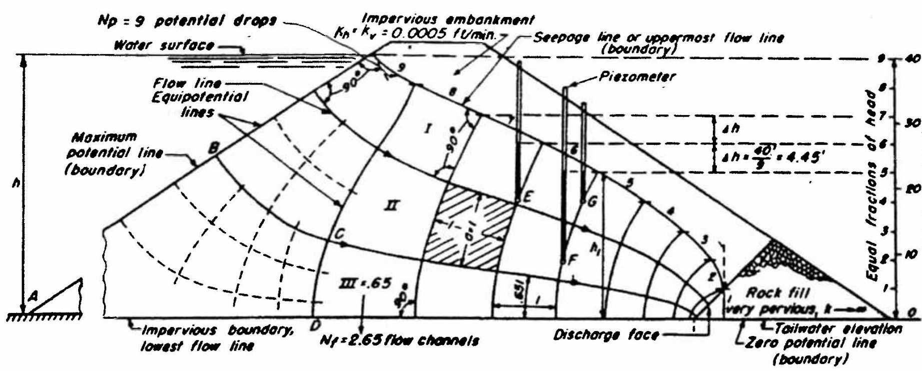
Prior to the advent of numerical analysis computer programs, physical models that scaled or simulated the flow of water through porous media were sometimes used to evaluate seepage behavior. Such models included electrical analogy, sand models, and viscous flow models. Of these, electrical analogy methods tended to be more frequently used. The methods were effective because of the analogy between Ohm's Law and Darcy's Law and because the Laplace equation describes both electrical potential distribution in a conducting medium and hydraulic potential in a saturated porous medium. Since physical models are typically no longer used, no further details will be discussed.

8.4.4.3 Numerical Analyses

Numerical analyses as coded into computer programs are today the most widely used method to analyze seepage issues. The most commonly used programs in Reclamation seepage analyses are discussed in section 8.4.6.

8.4.5 Flow Nets

The flow net is a graphical procedure consisting of hydraulic potentials and flow direction in a 2-D, saturated, steady-state seepage system. Flow nets can be useful for estimating pore pressure, hydraulic gradient, and flow quantity when the system can be idealized into one or two uniform material zones and limited parameter variation is required. The basic properties of a flow net are illustrated in figure 8.4.5-1, while example flow net calculations are shown on figures 8.4.5-2 and 8.4.5-3 [17]. For anisotropic permeabilities, flow nets can be transformed to represent differences in vertical and horizontal values. Figure 8.4.5-3 illustrates the use of a transformed section.



COMPUTATIONS:

1. Discharge per unit width of section:

$$Q = kh\frac{N_f}{N_p} = .0005 \times 40 \times \frac{2.65}{9} = .0059 \text{ ft}^3/\text{min}$$

per ft. of embankment

2. Hydrostatic pressure at any point:

$h_1 = \frac{n_p}{N_p}h$ where n_p = number of potential drops between point and zero potential

At point E, $h_1 = \frac{6}{9} \times 40' = 26.7$ft.

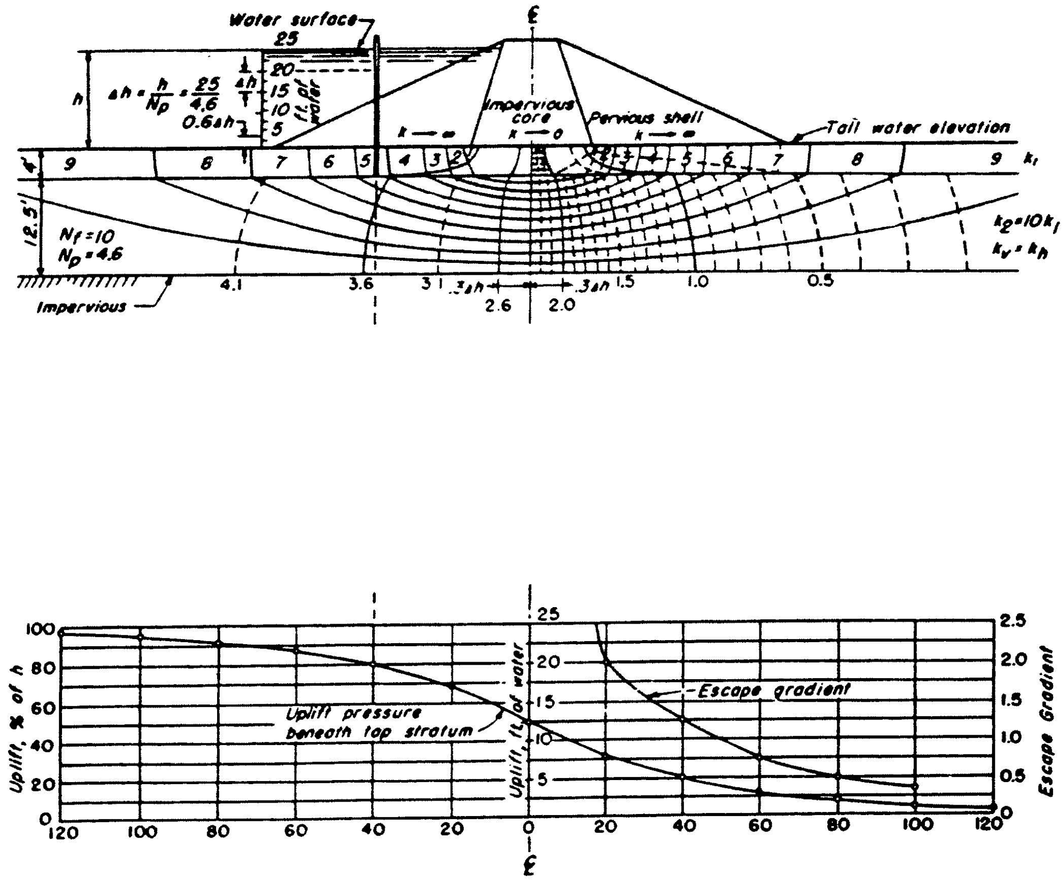
At point F and G, $h_1 = \frac{5}{9} \times 40' = 22.2$ft.

h_1 can also be determined graphically

3. Average hydraulic gradient for any "square" element:

$i = \frac{\Delta h}{l}$, for shaded area, $i = \frac{4.45'}{11.2'} = 0.40$

Figure 8.4.5-1. Typical flow net showing basic requirements and computations [17a].



$$Q = kh\,\frac{N_f}{N_p} = k_1\left(25\,\frac{10}{4.6}\right)$$

Figure 8.4.5-2. Seepage through foundation showing computations [17a].

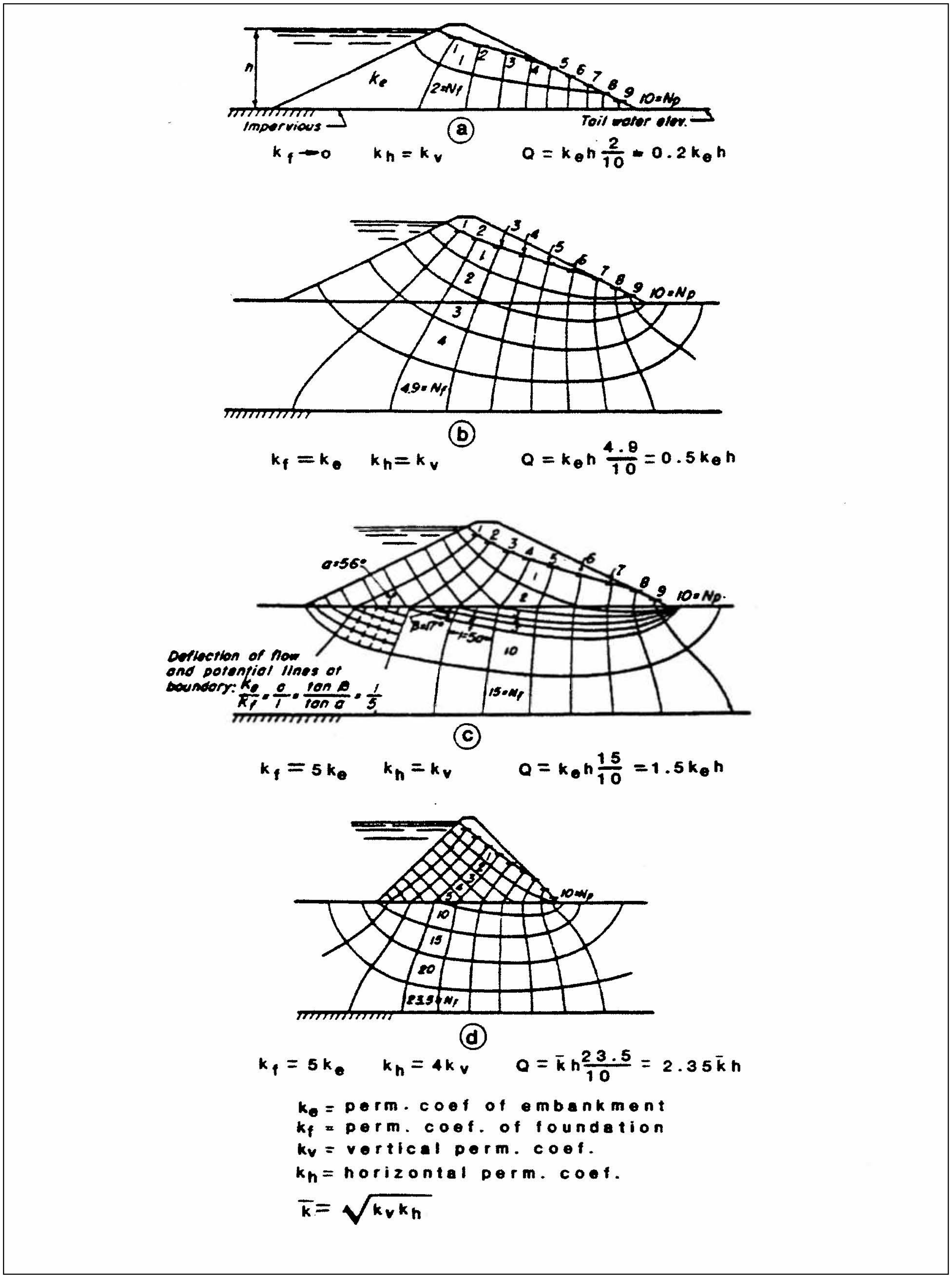


Figure 8.4.5-3. Seepage through embankment and foundation [17a].

Flow net experience helps in visualizing flow paths and gives the engineer insight into critical areas in a cross section. While flow nets are no longer commonly used for analysis, this method remains the best teaching tool for understanding the concepts of flow path and head loss through a dam. For less complex problems or for preliminary analyses, its use is still appropriate. The flow net is composed of two parts: the roughly vertical equipotential lines and the roughly horizontal flow lines. Drawing an accurate flow net requires experience due to the trial and error procedure of meeting the requirements for the graphical procedure (cell length equals cell height, and lines intersect at right angles). Engineers that will conduct seepage analyses are highly encouraged to read references on flow nets (such as Cedergren [3]) and develop a basic skill (or at least understanding) in graphical flow net construction so that this simplified method can be applied in appropriate situations.

This graphical procedure is generally best applied to homogeneous and simple systems because multiple anisotropic material zones or a requirement for parametric variations typically lead to the need for higher level analysis procedures.

8.4.6 Numerical Analyses

Numerical analyses, in the form of commercially available computer programs, have become increasingly sophisticated and are widely used to model a variety of seepage flow conditions and situations. Computerized numerical methods are recommended over other methods for all but the simplest seepage system modeling because of the following advantages:

1. Complex systems can be modeled relatively easily, although some simplification or generalized conceptualization of models may be necessary.

2. No transformation of dimensions or properties is necessary.

3. Since most numerical models use a finite element mesh, resultant properties or values (seepage flows, pore pressures, etc.) are available for each node or location within the model.

4. Parameter variation (sensitivity analysis) is much more expedient.

5. Results (pore pressures, phreatic surface) can be copied into stability programs.

6. Effects of adding or deleting design elements in an embankment dam can readily be seen.

Typically, seepage analyses are performed for the following reasons:

1. To estimate the phreatic surface within an embankment
2. To estimate pore pressures within an embankment or foundation
3. To estimate exit gradients and/or uplift pressures at the toe of an embankment
4. To estimate horizontal gradients through an embankment or foundation
5. To estimate the amount of seepage flow that may pass through an embankment or foundation
6. To evaluate the relative effectiveness of various seepage reduction measures
7. To estimate the amount of seepage flows intercepted by drainage features, such as toe drains or relief wells, and to size and optimize the configuration of these types of drainage features
8. To evaluate the effectiveness of, or to aid in the design of, dewatering systems

Not all of the seepage related issues and problems described in this chapter lend themselves to analytical or numerical analysis, especially when using 2-D analysis procedures described in this section. In Reclamation, 2-D seepage analysis is a norm; 3-D numerical models for seepage related issues have been used on some Reclamation projects (e.g., Virginia Smith Dam). Also, commonly used numerical models are based on solving the Laplace equation, $\nabla^2 \varphi = 0$, as a potential boundary value problem; φ is the potential function; ∇^2 is the Laplace operator: $\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$. For seepage problems, the potential function is the total head.

Following are seepage analysis programs that Reclamation has used (either in house or through contracting) to evaluate these type of seepage issues.

8.4.6.1 SEEP/W

SEEP/W is the primary seepage analysis program currently used by the Geotechnical Engineering groups. Part of the GeoStudio suite of engineering analysis applications, SEEP/W is a 2-D, finite element software program for analyzing ground water and excess pore-water pressure dissipation problems in a porous media. The comprehensive nature of the program enables analyses ranging from simple, saturated, steady state problems to sophisticated, saturated and unsaturated, time dependent problems. Good quality output graphics allow a visual display of equipotential lines and flow paths, and contours can be plotted for a number of properties/results such as pore pressures, seepage velocities, and gradients. As with most seepage analysis programs, computations include flow quantities and uplift pressures at user-selected locations in the model. Appendix C contains an expanded discussion of SEEP/W usage and includes example analyses.

8.4.6.2 FLAC

Fast Lagrangian Analysis of Continua (FLAC) is a 2-D, explicit finite difference program that can model a number of different engineering applications. Although it is most typically used within Reclamation for analysis of seismic deformations, it can also be used for seepage analyses. As a 2-D program, however, there appears to be little benefit in using it over a simpler program like SEEP/W. FLAC may be useful in modeling pore pressure effects on stability of an embankment. Since Reclamation has had little experience in using FLAC for seepage analyses, analysts should consult the program's user manual for information on potential applicability and whether it is the best tool for the job.

8.4.6.3 FRACMAN

FRACMAN is a program that models fracture networks in rock and, thus, permits the simulation of flow through fractured bedrock, as opposed to equivalent porous media models. Reclamation has had relatively limited experience with it. Obviously, a lot of geologic information is required in order to develop a reasonable model of the fractured/jointed bedrock system. The U.S. Geological Survey has more experience with the program and was contracted to model seepage through the bedrock foundation at Horsetooth Dam.

8.4.6.4 FRACK

Another approach to modeling flow through a fractured rock foundation is with the program FRACK. FRACK is a fractured media freeware flow and solute transport suite currently under development that is intended to serve as a preprocessor to MODFLOW when modeling flow through fractured media. The program can model both 2-D and 3-D fracture networks. FRACK is based on a fracture continuum method that closely approximates solutions to discrete fracture networks (DFN) by mapping fractures onto a computationally efficient finite-difference grid. The use of the grid allows for the solution of both matrix and fracture flow by the standard porous media flow simulator, MODFLOW. As a preprocessor, FRACK generates and maps networks of deterministic and/or stochastic fractures onto a regularly spaced finite difference grid, according to a fracture continuum method that closely approximates flow solutions to DFN simulations. Thus, the methodology employed by FRACK is applicable to field sites where the DFN simulation approach is valid—flow through a rock mass occurs exclusively through sparse to intermediate networks of interconnected fractures.

8.4.6.5 MODFLOW

MODFLOW has been used within Reclamation, particularly for the design of dewatering and unwatering systems. Outside of Reclamation, this program is widely used for evaluating 3-D ground water flow and contaminant transport simulations, in addition to well performance.

8.4.6.6 Boundary Integral Equation (BIE)

The boundary element method has been used in Reclamation for solving seepage related boundary value problems. It is an effective, efficient, and accurate method compared to other numerical methods discussed in this chapter. In this method, only the boundary of the flow region is discretized; thus a 2-D problem is reduced to a 1-D problem. The computer programs BIE2DCP and BIE2DCS are available in Reclamation for seepage analysis in zoned anisotropic medium.

8.4.7 General Seepage Conditions to Model

8.4.7.1 Steady State

The most common analysis performed for Reclamation seepage issues has been to model the effects on the embankment of an assumed steady-state condition of reservoir operation. The basic assumption for this condition is that the variable of time is not considered; instead, the reservoir head has been constant for sufficient time to result in a stable flow regime. In general, this usually results in a somewhat conservative analysis, since Reclamation reservoirs are typically used for irrigation and, thus, many of them experience significant annual fluctuations. Analyses of embankment piezometers have shown that some of our embankments have not reached a steady-state phreatic surface even after decades of operation. Given the uncertainties with assigning permeabilities and modeling seepage behavior, any conservatism inherent in the assumption of steady-state conditions is usually considered acceptable. Thus, when analyzing flow quantities, gradients, and pore pressures at an embankment under normal operating conditions, a steady-state analysis is appropriate. Appendix C contains some example problems using SEEP/W to evaluate steady-state seepage conditions.

For flood loadings, when the reservoir will typically be at elevated reservoir levels for only days or weeks, a steady state analysis at the elevated reservoir level is likely too conservative, and a transient analysis would be more appropriate and should be used.

8.4.7.2 Rapid Drawdown

Upstream slope failures can result from a rapid drawdown of the reservoir, which could lead to a removal of the buttressing effect of the reservoir and insufficient time for dissipation of pore pressures in the embankment. The combination of these two factors can lead to instability of the upstream slope. In most cases, this failure mechanism is not considered a dam safety risk because the lower reservoir and large freeboard make it very unlikely that the dam will breach or result in catastrophic discharge flows. However, the economical and/or operational consequences can be very substantial (as was the case at B.F. Sisk Dam). Therefore, an evaluation of rapid drawdown stability is usually important. Rather than using the potentially over-conservative assumption of no pore pressure dissipation, SEEP/W can be used to estimate the pore pressures that might remain or develop in the embankment during a drawdown of the reservoir. These pore pressure values can then be input into SLOPE/W to analyze the stability of the upstream slope.

8.4.7.3 Transient Flows

Examples of transient analyses include determining the effects of a short-lived flood loading on an embankment (or levee), and estimating how long it will take a saturation front to move through an embankment or foundation. SEEP/W has the capability to model flow through unsaturated portions of an embankment and foundation under these conditions. Appendix C includes an example SEEP/W evaluation of a transient analysis for a reservoir first filling behind a new embankment dam. Transient analysis can also be used to estimate how long it will take to achieve drawdown from dewatering wells.

8.4.7.4 Well Performance (Dewatering, Relief Wells, etc.)

Geotechnical engineers may be called upon to design a relief well system for a dam with potential confined flow concerns, or they may need to design a dewatering system to remove water from an embankment/foundation to be excavated. While this is obviously a 3-D problem concerned with both the depth and lateral extent of the area to be dewatered, well performance can often be modeled with a 2-D program such as SEEP/W. Instead of constructing a typical vertical section, a “plan view” is modeled with a uniform permeability and given depth assumed for the foundation. In this way, well spacing can be evaluated. Appendix C contains an example SEEP/W problem dealing with well performance.

8.4.8 Specific Seepage Issues to Model

8.4.8.1 Seepage Flow Volumes

An estimate of seepage flow volumes is often important for the design of seepage-related features, for comparison of the effectiveness of different design features, for determination of whether reservoir losses will adversely impact the performance of the facility, or for similar issues. For design of drainage collection systems, seepage analyses are used to provide a general order of magnitude estimate of expected inflows into drains; this can be used to select pipe sizes. Examples include estimates of toe drain flows or potential flows into relief wells.

Quantitative estimates of seepage flows are also useful in comparing various seepage reduction features in order to optimize the design. Examples include the optimum length of an upstream blanket or the optimum depth of a vertical cutoff wall, the impact of different drain locations (and depths), and the effectiveness of different width cores or even the location of the core.

Overall seepage flows obviously might be a consideration for a new dam or for a raised dam where an operational change is proposed that will result in a higher pool level.

For any seepage analysis where flows are being quantified and/or form the basis for design assumptions, it is important to remember the inherent uncertainties with these analyses and to apply conservative safety factors so that the designs will safely handle potentially significant increases in estimated seepage flows. It is also useful to perform sensitivity analyses, using varying assumptions for a key parameter such as permeability, to better understand the potential range of flows.

8.4.8.2 Exit Gradients and Uplift Potential

When inadequate piezometer coverage prevents a reliable estimate of vertical gradients at the toe of a dam, seepage analyses are used to estimate those gradients. As discussed in section 8.2.2, evaluation of exit gradients should be limited to those cases where the soils are cohesionless, while the evaluation of uplift is generally concerned with the case where a low permeability layer overlies a pervious, confined layer. These types of analyses can be very sensitive to permeability values, the presence of tailwater, and separate piezometric lines in underlying and overlying layers. Again, sensitivity analyses are useful in characterizing the potential range of gradients possible. In addition, be aware that the size of elements in numerical models makes a difference when calculating exit gradients. For more discussion of this issue, refer to reference [54].

When calculating factors of safety against uplift, care should be given in assigning unit weights to the overlying layer or any weighted berm placed as a mitigation measure, as well as the expected piezometric levels (if any) within the confining layer.

8.4.8.3 Internal Gradients

Although the initiation of internal erosion along a typically horizontal flow path is admittedly difficult to assess, seepage analyses may provide useful information in terms of the potential gradients along the seepage path. With SEEP/W, individual piezometric heads or gradients can be obtained from each node or element in the finite element mesh, which enables an analyst to view the estimated gradients throughout the embankment or foundation being modeled. In combination with viewing the estimated flow paths and understanding the properties of the soils in the flow path, the gradient values help provide insights into the potential for internal erosion to initiate and, perhaps, which locations might be critical.

8.4.9 Modeling Considerations

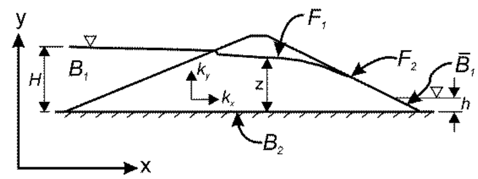
8.4.9.1 Model Size and Complexity

A standard approach for most seepage models is to ensure that the end boundaries of the model are sufficiently distant from the key features being modeled to ensure end effects do not come into play. This also helps ensure that supplemental design features (not originally envisioned) can be added at the upstream and downstream ends of a key feature. When first laying out the overall size of the model, consider extending it a bit farther than first estimated to be necessary – a model can be made smaller later on much easier than it can be made larger. In addition, given the speed of computers, a larger model is not a particular problem. One rule of thumb is to consider the footprint of the embankment (or similar feature) being evaluated, and then size the model to extend two additional footprints upstream and two additional footprints downstream.

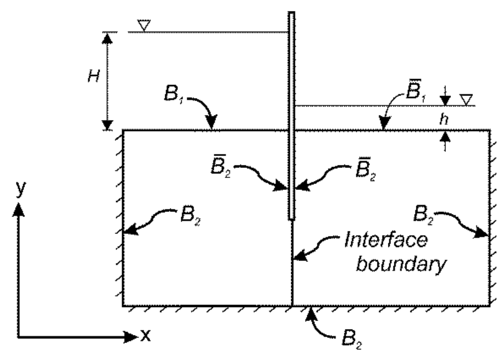
The overall size of a finite element mesh and the individual elements (such as utilized in SEEP/W) ideally reflects an optimum balance of sufficient size and elements to portray reasonable behavior and valid results, while also ensuring reasonable computational time. A good piece of advice is to start with a relatively simple model and add complexity as needed. Given the numerous variables and nonhomogeneity in most natural (and even manmade) soils, as well as the typical complexity of a groundwater or seepage regime, any model will ultimately be a simplified representation of actual conditions. There are rarely sufficient data to adequately define a geotechnical problem; that is simply the nature of our work. With this recognition, a good model condenses the critical components of a site into a relatively simple representation and does not include features that are not likely to be major contributors to the behavior being modeled (benign elements). An advantage of starting simple is that the analyst can easily check the model for errors, as well as develop a basic understanding of the system behavior. To start with, a finite element mesh can be relatively coarse. As the modeling continues, elements can be made smaller, and additional materials or geologic units can be defined, as needed, if additional detail would add to the understanding of the system behavior.

8.4.9.2 Boundary Conditions

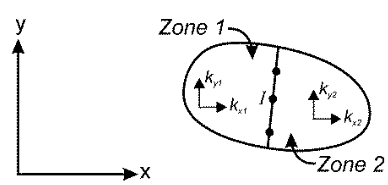
The establishment of boundary conditions is a key part of any seepage analysis and should be carefully considered. For seepage problems, the boundary conditions are in terms of total head or its gradient in a direction normal to the boundary. Typical boundary conditions for embankment dam and foundation problems are shown in figure 8.4.9.2-1 [55].



(a) Boundary conditions for a typical free-surface seepage through homogeneous embankment dam (unconfined flow problem).



(b) Boundary conditions for flow under sheet pile wall (a typical confined flow problem).



(c) Boundary conditions at interface between zoned homogeneous and anisotropic regions.

Governing differential equation for steady state seepage through a homogeneous, anisotropic medium:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial \phi}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial \phi}{\partial y}\right) = 0$$

Boundary conditions for unconfined flow, Figure (a):

Face	Prescribed boundary condition
B_1	$\phi = H$
$\bar{B}_1$	$\phi = h$
B_2	$\frac{\partial \phi}{\partial n} = 0$
F_1	$\phi = z$; $\frac{\partial \phi}{\partial n} = 0$
F_2	$\phi = Y$

Boundary conditions for confined flow, Figure (b):

B_1	$\phi = H$
$\bar{B}_1$	$\phi = h$
B_2	$\frac{\partial \phi}{\partial n} = 0$
$\bar{B}_2$	$\frac{\partial \phi}{\partial n} = 0$

Boundary conditions for interface, Figure (c):

In the transformed space with $X_T = \sqrt{\frac{k_y}{k_x}}\, X$ and $Y_T = Y$, the interface boundary conditions are:

$$\phi_{I_1} = \phi_{I_2}$$

$$\left[\sqrt{k_{x_1}\, k_{y_1}}\, \frac{\partial \phi}{\partial n}\right]_{I_1} = -\left[\sqrt{k_{x_2}\, k_{y_2}}\, \frac{\partial \phi}{\partial n}\right]_{I_2}$$

Figure 8.4.9.2-1. Boundary conditions [55].

Analysts are encouraged to carefully read user's manuals and other relevant information about the program being used, as well as ask advice from more experienced users, to ensure that boundary condition options are well understood and appropriate conditions are selected. In addition, it is advisable to study different hydraulic conditions (varying head).

8.4.9.3 Three-Dimensional Effects

Two-dimensional seepage models are the most typical method employed within the Geotechnical Engineering groups, given the convenience and relatively quick run times using the program SEEP/W. In addition, given the complexities and uncertainties of modeling seepage within an embankment and foundation, a strong case can be made for using a relatively simple model. However, it is important to recognize that some seepage problems may have some significant 3-D effects. Cases where these types of effects may be important include seepage and gradients at the ends of cutoff walls or at gaps or flaws, seepage into drains, and similar instances where a large area is contributing seepage flows that end up being concentrated in a small area.

In these cases, the analyst should recognize that a 2-D model may not fully represent the field conditions and may underrepresent the seepage flows. Therefore, a design may include some additional features or safety factors to allow for the potential of more seepage or higher gradients. For a more detailed analysis of a situation, it may be worthwhile to create two SEEP/W (2-D) models, one portraying a section view and one portraying a plan view, to gain a better understanding of flow components. In addition, permeabilities may be increased near drains and ends of walls to represent the potential of higher flows in these areas. Finally, if 3-D effects are considered potentially critical, a 3-D model using some of the programs briefly discussed in section 8.4.6 may be utilized.

8.4.9.4 Model Calibration and Sensitivity Studies

Given the uncertainty in soil and rock properties for most geotechnical evaluations, model calibration is an important step in any seepage analysis. Model calibration essentially consists of varying soil properties and reservoir loadings to determine how well the analysis results compare to known observations. This is particularly true for seepage analyses, as permeability can be very sensitive to variations in gradation or density for many materials. Conducting sensitivity, or parametric, studies should be a critical part of any seepage modeling effort. When analyzing an existing structure that has performance data, such as piezometric levels and seepage amounts, the sensitivity studies serve as a means of calibrating the seepage model to measured performance. In this case, parameters are varied, typically one at a time, until the modeled behavior closely approximates the measured behavior. There are multiple valid solutions for any given model where the number of unknowns exceeds the knowns. Therefore, it takes judgment, experience, and consideration of the reasonableness of each material/parameter property to select the ultimate values to use in the model. In addition, be aware that measured performance data may be suspect in some cases, whether due to instrument error, an inability to locate instruments in the most critical areas, or simply insufficient instrumentation. In other words, consider whether piezometers and seepage measurement devices are fully capturing the expected behavior of an embankment or foundation.

Sensitivity studies are equally important for modeling new or existing projects without performance data because they are critical to a better understanding of potential seepage behavior. Proper sensitivity studies are involved and well thought out. Numerous simulations are performed by varying parameters, one at a time, and results are compared and interpreted. It should not be a hasty exercise but should be a key aspect of the overall modeling effort. A thorough sensitivity study should provide the analyst with a better understanding of which variables most affect the system behavior. This, in turn, allows the opportunity to refine those values or model a range of values to best portray the potential seepage-related behavior of the structure. It is important to remember that it is unlikely that one best answer will be found, but more likely that the analysis will predict a range of potential behavior. Avoid placing complete trust in analytical results; rather, consider the results as an indicator of expected behavior and test their reasonableness in terms of experience and engineering judgment.

A good example of a detailed model calibration can be found in reference [56].

8.4.10 Using Analysis Results for Design and Decisionmaking

There is little question that seepage analyses can provide valuable insights toward the understanding of potential seepage issues. However, analysis results should be used with an understanding of the uncertainties involved in seepage modeling. Although there can be considerable uncertainty in many (if not most) geotechnical engineering problems, the modeling and prediction of seepage behavior are arguably subject to the most uncertainty. Hence, results should be carefully considered while formulating actions to take in resolving seepage-related issues.

8.4.10.1 Degree of Accuracy in Analyses

Seepage analysts, as well as end users of analysis results, are encouraged to read the discussion on page 20 in Cedergren [3], where the author discusses the accuracy of seepage analyses. In essence, Cedergren notes that seepage analyses should probably be viewed as capable of predicting the general order of magnitude of results and approximating seepage behavior. Even when detailed numerical analyses are carefully performed, the results are only as good as the initial assumptions of material properties such as permeability (including anisotropy) and of the representation and understanding of embankment and foundation (geologic) conditions. Sufficient data to thoroughly model all key portions of a geotechnical environment are rarely available. Hence, Cedergren recommends that experience and common sense are essential in developing the model and interpreting the results or findings.

8.4.10.2 Consideration of Safety Factors

When required safety factors are listed in this standard (e.g., for such conditions as high exit gradients or uplift pressures), the required factors are viewed to be reasonable to conservative in order to provide assurance of no failure. However, it should be recognized that an abundance of quality data and model confidence could permit a lowering of the required safety factor. For example, a good array of piezometers which enable a thorough understanding of pressures or gradients beneath a dam could justify the use of a lower required safety factor due to less uncertainty in the analysis. Although this may be acceptable for a particular site, the lowering of required safety factors is usually not done because seepage analyses typically still contain significant uncertainties.

8.4.10.3 Potential Different Applications for New and Existing Embankments

For new dams, there is rarely any performance data on which to justify a reduction in required safety factors, so designs of new dams or modifications to dams should strive to meet the recommended factor of safety and follow practices described in this standard. All potential failure modes should be considered in the design of a new structure or modifications to an existing structure. Analysis of seepage issues is appropriate, but ultimately, the designs should ensure that all potential seepage is controlled by appropriate design features. The analyses may help to identify areas of special attention (such as at the abutments or beneath cutoffs), provide insights into the relative amounts of seepage to be handled (which can aid in the sizing of drainage features), and indicate potential hydraulic gradients (which may lead to design changes to minimize gradients).

In Reclamation dam safety evaluations of existing dams, analyses tend to provide probabilistic scenarios and results, rather than deterministic safety factors. Strictly deterministic safety factors or analysis results are typically put in probabilistic terms for risk analysis [7]. Existing structures have also typically been tested under decades of reservoir operation, which can provide empirical evidence of seepage behavior (as opposed to theoretical behavior assumed for new and untested dams). Hence, safety factors **might** be relaxed and/or a higher degree of credibility given to analysis results for cases where good data and years of successful operation are apparent. **However, it is always important not to place undue confidence in years of successful performance because Reclamation experience has shown that internal erosion incidents can suddenly manifest after decades of previously satisfactory performance.**

8.4.10.4 Use as Input for Risk Analysis

Predicting probabilities of failures for seepage-related failure modes, including internal erosion, is difficult, particularly given the large number of uncertainties involved in material permeabilities, actual hydraulic gradients, precise location of seepage flows, and similar variables. In order to help the process, risk teams rely on both actual performance data and seepage analysis results to make estimates of structural response to seepage. In particular, teams will look for guidance on expected hydraulic gradients in various portions of the embankment or foundation, expected pore pressures, seepage velocities, seepage flows, and any locations of potentially anomalous seepage patterns. A material's resistance to erosion is also quite important. Calculated safety factors or quantitative estimates of flows, or other parameters, may not be as important as insights into relative behavior (or trends) in certain areas of the dam/foundation or predicted behavior at the dam as compared to other dams.

8.5 Seepage Mitigation Measures

8.5.1 General

This section of the design standard discusses various actions or design features typically used to mitigate seepage concerns, whether at new dams or at existing dams. Such features or measures can be categorized into two general types: seepage control and seepage reduction. A fundamental aspect of embankment dam design is the use of multiple defenses to ensure safety. Thus, it is typical to see a combination of seepage mitigation features incorporated into a well-designed dam. This design approach may be even more important with seepage issues given the wide range of variables and uncertainties associated with seepage flow through embankments and their foundations. Reclamation designs of new dams or modifications to existing dams should always follow the philosophy of multiple lines of defense.

8.5.2 Seepage Control Measures

Seepage control measures aim to collect or direct seepage into engineered features, where it can be controlled to minimize the development of adverse behavior such as high gradients, excessive pore pressures, large seepage flows, or similar problems. In general, these methods focus on proper filtering and drainage of seepage flows.

8.5.2.1 Embankment Internal Filter or Drain

Internal filter and drainage features for an embankment dam typically include a chimney filter and/or drain located immediately downstream of the core of the dam, connected to a horizontal filter and/or drainage blanket that extends to the downstream toe of the dam. Quite often, this filter and/or drain system is comprised of two separate zones to ensure both filter compatibility and adequate drainage capacity. The use of upstream-downstream oriented pipes within an embankment to enhance horizontal drainage always should be avoided, given the potential to introduce a transverse defect in the embankment and the difficulty in making future repairs or even inspections. Similarly, the use of geosynthetics as critical filters or drains within an embankment is discouraged. Natural, processed sands and gravels serve as the best internal filter and drain components. Both the chimney and blanket portions of the filter are designed to ensure that finer materials in the core or foundation cannot erode into downstream zones. Filters and drains should extend deep enough in the foundation and high enough in the embankment to ensure that all potential pathways for internal erosion are properly protected. The internal filter or drain system is unquestionably one of the most important aspects of the design of an embankment dam and should be carefully designed and constructed. The proper design of filters and drains is discussed in detail in Reclamation's *Design Standards No. 13, Embankment Dams*, Chapter 5, "Protective Filters."

8.5.2.2 Toe Drains

Toe drains typically serve as the collection system for the internal drainage system in the embankment, as well as a drainage source for foundation seepage. As such, toe drains need to be carefully designed to fully satisfy filter criteria for both embankment and foundation soils. Toe drains typically consist of perforated or slotted pipe surrounded by a gravel or small rock envelope which, in turn, is surrounded by filter sand or gravel. The design of filter protection for toe drains is described in Reclamation's *Design Standards No. 13, Embankment Dams,* Chapter 5, "Protective Filters."

The toe drain pipe should be sized to safely accommodate the amount of expected seepage. Seepage analyses are often used to estimate the flow amounts, although pipes should be sized to comfortably handle more than the estimated flows given the uncertainties in analysis. The toe drain is normally placed as low in the embankment as the discharge point and downstream topography will allow in order to provide maximum drainage. Inspection wells constructed along the toe drain provide access to the system for inspection, monitoring, and maintenance. Design considerations for toe drain systems are discussed in more detail in Reclamation's *Drainage for Dams and Associated Structures* [57], *Design of Small Dams* [44], and appendix E of *Design Standards No. 13, Embankment Dams,* Chapter 5, "Protective Filters."

Another important advantage of toe drains is that they provide a means for quantitative measurement of seepage to aid in observation/analysis of seepage-related behavior. As such, a flow measuring device such as a weir or flume is typically included at one or more locations within a toe drain system.

8.5.2.3 Drainage Trenches

Downstream drainage trenches running parallel to the toe of the dam can be used when downstream drainage of the foundation is needed beyond what is normally provided by a toe drain. In essence, the deeper trenches provide relief of pressures and a filtered outlet for seepage layers that are located at a greater depth than would be encountered with a typical toe drain. Trenches are excavated and filled with filter/drainage materials of specified gradation to prevent piping of adjacent foundation soils into the trench. As with a toe drain, a perforated or slotted collector pipe is typically included and set at the lowest possible elevation that will still allow downstream outfall. Special machines for agricultural drain installation can be used; these machines excavate the trench, brace the open excavation, and allow for pipe installation and backfilling (as at Bonny Dam and Pablo Dam). Reclamation has also constructed trench drains using slurry trench methods with biodegradable slurries (as at Wasco Dam). For use at existing dams, the stability of the excavated and backfilled trench should be evaluated.

8.5.2.4 Relief Wells

Relief wells are used to reduce excessive pore pressures in pervious foundations to a tolerable level. Relief wells provide safety against high exit gradients or uplift pressures. Frequently, relief wells are used to reduce artesian pressures in confined aquifers. Carefully designed "filter packs" are placed around the well screen to ensure that foundation materials are not piped into the wells. Design considerations for these types of wells are discussed in Reclamation's *Ground Water Manual* [15] and the USACE's *Relief Well Manual* [58].

Potential shortcomings of relief wells may include relatively poor efficiency in seepage collection due to their small influence area and the potential for clogging over time (such as occurred at Red Willow Dam). A periodic program of well cleaning and development over the years is often required with this design element. Flows at full reservoir should be measured and documented. Relief wells may only operate during flood surcharge, and their performance may be unknown until that time. Contingency plans could include placing pumps in the wells to improve performance.

8.5.2.5 Horizontal Drains

Horizontal or semi-horizontal drains can be bored into foundations (frequently in abutment areas) to relieve excessive pore pressures or intercept seepage. Horizontal drains have been constructed in both rock and soil materials. Careful attention to screening and filtering is essential to prevent the potential for internal erosion into the drains. These types of drains were used at Joes Valley, Red Fleet, and Costilla Dams to lower high pore pressures and seepage gradients in the abutments.

8.5.2.6 Drainage Galleries and Tunnels

These features consist of formed drainage galleries at the base of an embankment or tunnels bored into the foundation from which a series of drain holes fan out into the foundation (which typically is rock). In some cases with erodible rock, it may be necessary to line the tunnels. The general intent of these features is to relieve pore pressures and remove and control seepage flows from beneath the embankment, often with a focus at the embankment or foundation contact. Galleries can also sometimes be used to grout the foundation. An example of a drainage tunnel at a Reclamation embankment dam as part of the original construction is Ridgway Dam, where a part-tunnel (into foundation), part-conduit (through embankment) was constructed and completed with an extensive array of drainage holes to intercept seepage through the left abutment and lower water levels. Similar features were constructed at Soldier Creek and Navajo Dams after original construction. A caution on using this design feature is the potentially high gradients that might develop in the soils surrounding the drains. For this reason, drains that are installed near the embankment contact or in erodible rock should be filtered to minimize any potential for piping of soils into the drains. Periodic observations of these drains are necessary to check for any evidence of sediment transport.

8.5.2.7 Structure Underdrains

Spillway and outlet works designers will typically include structure underdrains under chutes and stilling basins to reduce uplift pressures and stagnation pressures, which could lead to slab jacking. Underdrains on competent rock foundations are rarely problematic. However, experience with underdrains for structures with soil-like foundations at several existing Reclamation dams has shown that damaged pipes or improperly designed filter envelopes can lead to piping of backfill or foundation materials into the drains. This condition not only endangers the overlying structure, but it also creates the potential for the internal erosion to progress upstream beneath/through the embankment and, ultimately, leads to breach of the dam. Designers of new or replacement structures should be especially careful about ensuring that properly graded filters surround the underdrains and that the drains are carefully protected during the construction phase. Due to the inability to access these features after construction, underdrains should not be wrapped with geotextile.

8.5.2.8 Conduit Filter Envelopes

Historically, concentrated seepage along conduits has been one of the main contributors to internal erosion failures in embankment dams. Prior to the mid-1970s, conduits within Reclamation embankments typically included cutoff collars to increase the length of seepage pathways along the outside of the penetrating conduits. Since that time, however, cutoff collars have been eliminated from designs because of concerns that they complicate compaction efforts and potentially create adverse stress concentrations. Instead of cutoff collars, modern conduits feature battered walls to facilitate compaction of embankment materials. In addition, excavations for conduits are made wide enough with reasonable slopes to help achieve adequate compaction and minimize the chance for arching of embankment soils over the excavation. However, the key feature in protecting against internal erosion along conduits is a filter envelope that fully surrounds the conduit and is in intimate contact with it. Often, this filter envelope may be a part of the internal chimney filter or drain in the embankment, as described in section 8.5.2.1. In order to place a filter beneath an existing conduit underlain by potential erodible materials, a short section of the conduit may have to be removed and replaced as was done at Keechelus Dam and Caldwell Canal at Deer Flat Dams. New designs or modifications to existing dams should include filter envelopes around any conduit that penetrates an embankment. This policy is described in reference [59].

8.5.2.9 Foundation Surface Treatment

Since one potential unfiltered exit is at the embankment/bedrock contact (typically at the base of the cutoff trench), appropriate foundation treatment measures are needed to ensure embankment core materials cannot erode into bedrock discontinuities. Such treatment measures include the placement of slush grout to fill in open cracks at the rock surface, dental concrete to cover fractured/jointed areas and smooth out foundation irregularities, and the use of filters on areas of the rock surface that are highly deteriorated or fractured/jointed.

The design of foundation treatment measures is addressed in Reclamation's *Design Standards No. 13, Embankment Dams*, Chapter 3, "Foundation Surface Treatment."

8.5.3 Seepage Reduction Measures

There are a number of different seepage reduction measures, with almost all of them essentially reducing seepage by means of extending the seepage path through the use of vertical or horizontal barriers. This lengthening of the seepage path results in a lowering of the hydraulic gradient and, thus, a reduction in seepage flows.

8.5.3.1 Embankment Core and Location

The effectiveness of a wide embankment core acting as a seepage barrier should not be underestimated. Due to low gradients through wide cores, seepage is minimized. Wide cores have been a feature of Reclamation dams for decades and may help explain why the older dams designed without chimney filters or drains do not experience internal erosion through the embankment. Wide cores of relatively impervious soils lead to significant head losses as the seepage traverses a long path. In addition, a wide core reduces the chance that any defect in an embankment will create a seepage path that is continuous. For that reason, past Reclamation guidance typically has been to limit the width of the core to no less than one-fourth to one-third the reservoir head. Thinner cores can be used; however, thinner cores lead to higher gradients through the core and place an even greater reliance on the filter compatibility of adjacent filter or drain and transition zones.

The location of the core varies in Reclamation embankments. Most commonly, the core is located in the center of the embankment, which has the advantage of providing the highest contact pressure at the base of the core and typically leads to a cutoff trench located in the center of the dam. The placement tends to enhance slope stability for dams that have a weak foundation layer left in place (by limiting the extent of both upstream and downstream failure surfaces passing through the foundation). However, sloping upstream cores also have advantages, including reducing head further upstream (which minimizes gradients downstream) and providing a large unsaturated zone downstream, which improves stability during seismic loading. Regardless of location, the top elevation of the core should be taken to at least the highest reservoir level expected during the design flood.

The design of embankment zoning is addressed in more detail in Reclamation's *Design Standards No. 13, Embankment Dams*, Chapter 2, "Embankment Design."

8.5.3.2 Cutoff Trenches

A well-constructed cutoff trench located beneath the core of a dam and backfilled with impermeable soils is a very reliable means of minimizing seepage through pervious foundation soils. In addition, since the excavation of this feature enables a complete view of foundation conditions, it enables a designer to gain firsthand knowledge of the foundation materials, provides the ability to adjust the design (for example, filter gradations) if needed, and permits foundation treatment at the bottom of the excavation and filter protection along the downstream face of the excavation. Cutoff trenches, if practicable, should be fully penetrating to an impervious zone and should have a wide base and no overly steep side slopes. Narrow cutoff trenches with steep side slopes raise the potential that soils within the embankment will "arch" across the trench. If this happens, the soils within the cutoff do not experience the full weight of the overlying embankment and may be susceptible to cracking from hydraulic fracture. Partially penetrating cutoff trenches are not nearly as effective in reducing seepage as fully penetrating trenches and are typically used only if they can be keyed into a low permeability layer within the foundation, or if the depth to rock is excessive. Refer to the charts in appendix B for a relative comparison of the effectiveness of different cutoff depths.

The design of cutoff trenches is addressed in more detail in Reclamation's *Design Standards No. 13, Embankment Dams*, Chapter 2, "Embankment Design."

8.5.3.3 Slurry Trench Cutoff Walls

Cutoff walls constructed by slurry trench methods can effectively cut off seepage in the embankment and/or foundation of dams. For new dams, slurry trench cutoff walls have been used as the impermeable water barrier for an embankment (instead of an impervious earth core) or as a foundation cutoff when the bedrock (or other suitable impermeable layer) is relatively deep, making a traditional cutoff trench excavation very costly. On existing dams, slurry trench cutoff walls have been used to reduce seepage through embankments, soil foundations, and rock foundations.

These features are constructed by excavating relatively narrow trenches, typically 2 to 5 feet in width, with bentonite slurry pumped into the excavation to support the trench side walls and prevent collapse during construction. The relative impermeability of a slurry cutoff wall results, in part, from the slurry forming a filter cake against both side walls of the trench. To keep the slurry approximately level and within a couple of feet from the top of the excavated trench, the working surface must be kept level. For relatively level ground (and depending on the nature of the backfill), the trench can be kept open for a significant distance. On sloping ground, a series of stepped working surfaces is needed, and increments of the wall are constructed separately with overlaps into previously constructed segments. Equipment used to excavate these cutoff walls can include large backhoes, draglines, clamshells, and specially constructed rock milling machines designed to cut through rock as well as soil. Slurry trenches have been excavated to depths of approximately 400 feet or more, which is considered by some in the industry to be a practical limit. However, specialty contractors claim to have capability to go deeper, and refinements in technology may lead to greater depths being common.

A variety of backfill materials can be used to construct the final wall. Originally, slurry trench cutoff walls were typically constructed of soil-bentonite backfill. The excavated soils, usually saturated with slurry, are cast to the side of the trench. These materials are then sluiced with more bentonite slurry; additional fines are added, if needed, to help ensure low permeability; and the materials are then worked with dozers to produce a well-mixed, soil-bentonite backfill. This backfill is then dozed back into the trench, where it forms a sloping backfill that follows behind the excavation operation. An example of this feature at a Reclamation facility is a cutoff wall located within the upstream blanket at Virginia Smith (Calamus) Dam, which was built during original construction of the dam to minimize foundation seepage.

Another slurry cutoff wall method has been to add cement to the bentonite slurry to form a low strength backfill with no soil component. In this method, the bentonite slurry is typically mixed first within a slurry pond or large tanks until the bentonite slurry is fully hydrated. Dry cement is then added to the fully hydrated bentonite slurry by meter as the bentonite is pumped for delivery to the site of the cutoff trench. Because cement-bentonite slurry mixture has nearly the same density as the traditional bentonite slurry (and would not be able to displace the bentonite slurry by tremie methods), the cement-bentonite slurry is used to stabilize the trench walls during excavation and is left in the trench to form the slurry cutoff wall. Ultimately, the cement-bentonite mixture hardens, forming a wall with a 28-day unconfined compressive strength estimated to typically range from 10 to 30 lb/in^2, depending on cement content. This type of wall forms the diaphragm for Reclamation's Diamond Creek Dike and was used for the repairs for the modification of Reclamation's A.V. Watkins Dam.

One of the limitations of these two types of slurry trench cutoff walls is the low strength of the backfill. Because of the narrow trench, significant arching occurs such that the backfill typically does not experience the weight of the overlying soil and, thus, is in a low stress condition. This makes soil-bentonite, and cement-bentonite to a lesser extent, subject to hydraulic fracturing. In fact, Reclamation experienced hydraulic fracturing of the soil-bentonite slurry trench cutoff wall at Virginia Smith (Calamus) Dam while it was being constructed (with no reservoir load). Another concern with these types of walls is blowout under high gradient. If a slurry trench cutoff wall is relatively impermeable, there may be a high gradient across the trench. If a cutoff wall intercepts a pervious coarse zone, there is the potential that the high gradients could initiate piping of the backfill into the coarse foundation layer downstream.

To improve the resistance of the backfill to hydraulic fracturing or blowout, cement can be added to a soil-bentonite mixture to create a soil-cement-bentonite backfill. Mixing this backfill becomes more complicated than in a traditional soil-bentonite operation and may require a pugmill. Reclamation's Twin Buttes Dam Modification is an example where this type of cutoff wall was constructed.

Another type of cutoff wall constructed by slurry trench methods is the concrete diaphragm wall. With this method, the trench is usually excavated in panels, and then the slurry is displaced by tremmied concrete. A variation of this method is the use of "plastic" concrete which has a bentonite component and is thought to be less brittle than conventional concrete and more compatible with the surrounding soils. A plastic concrete cutoff wall was constructed at Reclamation's Meeks Cabin Dam. For additional strength, reinforcement steel cages can be constructed and lowered into the excavation prior to concrete placement to create a reinforced concrete wall. Unreinforced concrete cutoff walls were constructed at Reclamation's Navajo and Fontenelle Dams as part of dam safety modifications and as a component of the original design and construction at New Waddell Dam.

There are two typical locations for a slurry trench cutoff wall in an embankment: at the upstream toe or through the crest. For the upstream location, the cutoff wall typically ties into an upstream blanket. Advantages of this location include reducing gradients, pore pressures, and seepage flows beneath most of the embankment; separating the main components of the work so as to expedite or optimize the construction schedule for a new dam; providing a wider working surface; creating the possibility of future repairs if the reservoir could be drawn down; and keeping a potentially low strength vertical element considerably away from most of the embankment. When the slurry wall is located through the crest of the dam, which tends to be the more common location when modifying a dam, it has the significant advantage of minimizing both foundation and embankment seepage.

For both new and existing dams, seepage analyses can be used to model the potential effectiveness of these features and help determine the optimum locations, depths, and extent of the walls. For preliminary evaluations, the charts in appendix B may provide insight into the potential effectiveness of different penetration depths.

In the design of slurry trench cutoff walls, it is important to recognize that very high gradients will exist across these thin walls, at the base of the wall, and at the ends of the walls. Consequently, special attention must be paid to ensuring that the walls are founded in competent materials that will be able to withstand the potential erosive effects of high gradients and flow concentrations at these locations. When this type of feature ties into an existing embankment core, additional care should be taken to ensure that this connection is also well protected against internal erosion.

8.5.3.4 Other Types of Walls

In addition to slurry trench cutoff walls, there are several other types of walls that can be designed and constructed to serve as vertical seepage barriers in embankment dams. These wall types include sheet piles, secant pile walls, walls constructed of stiff geomembrane panels, and jet grouted or soil mixing walls.

Early pile walls in embankments consisted of timber; occasionally, an older dam with one of these walls will be encountered. However, timber pile walls are rare. Rolled steel is the most typical type of sheet pile wall, while vinyl and composite (such as fiber reinforced polymer) sheet piles are a relatively new development. These products consist of individual panels of various weights, stiffness, and cross-sectional configuration. A key feature is the interlocking joints along the edges of the sheets, which allow a continuous wall to be formed. The sheets are typically driven and sometimes vibrated into the ground by special equipment. Jetting is sometimes used to facilitate penetration. Sheet piles can be an effective and economical means of constructing a cutoff wall, particularly at relatively shallow depths and if located in soils with a minimum of large size particles. However, these types of walls do have limitations and difficulties, and they may not be suitable for use as a permanent critical or sole line of defense against seepage. Dense soils or soils containing cobbles or larger sized materials can damage the piles or create difficulties in achieving effective interlocks during installation. It is common to see some leakage at the interlocks; however, over time, these joints tend to seal somewhat due to corrosion (for steel) or migration of fines. Reclamation designed composite sheet pile walls to minimize seepage at Tarheel Dam and Fourth Creek Dam, which are small Bureau of Indian Affairs facilities in Oregon.

Secant pile cutoff walls are not nearly as common as other walls but can be considered as a potential means of constructing cutoff walls. These walls consist of circular columns excavated and then backfilled with concrete. By overlapping and keying in adjacent columns, a continuous wall can be constructed. A secant pile cutoff wall was constructed at Reclamation's Lake Tahoe Dam to minimize seepage through the small embankment wing dams.

Reclamation constructed a rather unique cutoff wall at Reach 11 Dikes. It consisted of both an impermeable barrier and a sand filter to mitigate the potential for internal erosion failure of flood protection dikes in the Arizona desert. A trench was excavated through the crests of the embankments, supported by biodegradable slurry. Stiff geomembrane panels of 80 mil high-density polyethylene were lowered into the trench on a steel frame system. Interlocks similar to those on sheet piles enabled the construction of a continuous wall. The frame was removed, leaving the membrane behind, and the trench was then backfilled with filter sand (by tremie pipe) as an extra measure of protection against internal erosion. In principle, the biodegradable slurry completely decomposes and leaves no trace of an impermeable filter cake, thus permitting a filter to be an effective second line of defense.

Jet grouted columns and soil mixing methods have been used as foundation improvement methods to treat foundations subject to liquefaction. However, they are sometimes considered as a seepage reduction alternative. The jet grouting method consists of inserting a special injection pipe into the ground to the desired bottom of the treatment. The pipe is slowly raised, while it simultaneously rotates and injects a grout mixture into the foundation soils, creating a grouted column. Soil mixing methods use similar techniques to create "cemented" columns or barriers. By overlapping these columns or putting in a closely spaced grid of columns, most of the foundation can be treated. In general, these types of walls would pose concerns as a sole defensive measure to reduce seepage or prevent internal erosion. That is because it is envisioned that a fully continuous wall is difficult to achieve; there may be some chance that a "window" exists. Reclamation has not yet used this type of wall as a seepage reduction measure.

The design of slurry trench cutoff walls and other types of seepage reduction walls is addressed in more detail in Reclamation's *Design Standards No. 13, Embankment Dams*, Chapter 16, "Cutoff Walls."

8.5.3.5 Grout Curtains

Grout curtains have often been used to reduce seepage through foundation and abutment rock, but as a seepage cutoff feature, their effectiveness varies greatly depending on geologic conditions. Although grouting can be dependable for reducing total seepage flow through the foundation, a single "window" in the curtain can allow a shorter flow path with concentrated seepage. The effectiveness may be increased by use of multiple grout lines. Neat cement grout is most commonly used in Reclamation applications and is generally reserved for grouting in rock foundations containing joints and fractures. A detailed discussion of grouting can be found in reference [60]. Similar to the caution on cutoff walls, a successful grout curtain can lead to very high gradients across the top of the grout cap at the embankment/foundation contact. Careful foundation treatment measures such as slush grouting, dental concrete, blanket grouting, and foundation filters (both at the base of the core, as well as at the downstream face of the cutoff trench) are necessary to ensure that no unfiltered exits for seepage exist that may have the potential to cause erosion of the core.

Grouting is addressed in more detail in Reclamation's *Design Standards No. 13, Embankment Dams,* Chapter 15, "Foundation Grouting."

8.5.3.6 Upstream Blankets

Upstream blankets are a horizontal extension of the embankment water barrier (usually an earthfill core) typically used at a site underlain by high permeability foundation materials that are too deep to allow economical construction of a fully penetrating cutoff. As with the seepage reduction measures discussed previously, this feature is geared toward lengthening the seepage path in the foundation.

Relatively impermeable soil materials are frequently used in an upstream blanket, although geomembranes can be an economical alternative. Because a high gradient will typically occur across an upstream blanket, it is important to ensure that blanket materials cannot pipe into the underlying foundation. This can be accomplished by designing a transition or filter material beneath the impermeable soil that meets filter criteria for the blanket and the foundation. The use of a geomembrane instead of low permeability soil will usually eliminate the need for an underlying filter, although a bedding layer and a protective cover will be needed to protect the geomembrane both during construction and throughout future operation. Since an upstream blanket is constructed of low permeability materials, it does not have to be particularly thick. The length to which the blanket extends upstream is generally more important and can be assessed by numerical seepage analysis. For preliminary evaluations, some of the charts included in appendix B can provide useful insights into the relative effectiveness of different length blankets. Design considerations for upstream blankets can be found in Reclamation's *Design of Small Dams* [44]. Design considerations for geomembranes can be found in Chapter 20, "Geomembranes," of this design standard No. 13.

8.5.3.7 Flat Slopes and Berms

The use of flat outer embankment slopes and berms can be an effective way of lengthening the seepage path through an embankment or its foundation and, thus, reducing seepage. In addition, downstream berms provide a means of increasing safety factors against uplift or instability due to high pore pressures in the foundation. Downstream berms also can function as seepage control measures when filters and drains are incorporated into their design.

8.5.3.8 Foundation Surface Treatment

Foundation surface treatment was discussed previously in section 8.5.2.9 as a means of controlling seepage by sealing unfiltered exits. However, some surface treatments can also serve as seepage reduction measures. Two such examples include blanket grouting and core walls. Blanket grouting is a newer method, and core walls are an older method. Blanket grouting is primarily used to consolidate the upper portion of the bedrock surface beneath a modern embankment core, using a large number of closely spaced holes with typical grouting depths of about 30 feet. In addition to consolidating and improving the competence of the rock foundation, blanket grouting can also reduce the permeability in the upper surface, thus reducing the potential for seepage at the embankment or foundation contact. Depending on the foundation condition, blanket grouting may be extended downstream of the core.

Core walls serve a similar purpose and were a common feature in earlier Reclamation dams. Essentially, a concrete wall was constructed at the base of a cutoff trench to create a seepage barrier at the embankment/foundation contact. This wall would typically extend at least 5 feet into foundation rock and extend about 8 feet or more into the overlying embankment core. These core walls were derived from the same philosophy as seepage collars used on a penetrating conduit; these barriers interrupt and lengthen the seepage path at the core or foundation contact. However, given the unreliability of these walls and the issues of stress concentrations and difficult compaction, these features should not be used on new dams.

8.5.4 Temporary Emergency Measures

When seepage problems are identified, it is generally prudent to take immediate action because uncontrolled seepage can quickly lead to significant internal erosion and possible dam failure.

8.5.4.1 Pool Drawdown

If achievable within a sufficiently rapid timeframe, lowering the reservoir is probably the most effective emergency action to address a seepage problem. A lowered pool both reduces the head and hydraulic gradient, and creates more freeboard, making a complete dam breach less likely. Reservoir drawdown has invariably been the first step Reclamation takes when a developing seepage or internal erosion problem is observed to be potentially serious. The effectiveness of this action typically depends on how quickly the reservoir can be lowered, which is a function of reservoir volume and outlet release capacities. An example of successful reservoir drawdown that saved a dam from an internal erosion failure would be Fontenelle Dam in 1965.

8.5.4.2 Emergency Filters

Internal erosion often continues to progress and enlarge if there is an open or unfiltered exit point for the seepage. Consequently, when extensive and muddy seepage is observed discharging at a downstream location, efforts should be made to cover that seepage with filter material. In emergencies, it is not critical to locate and use an ideal filter material; rather, any type of suitable material close to meeting filter material should be located. C33 sand and standard concrete aggregate mixes are often suitable; crest surfacing and similar embankment zones may also be considered in an emergency. The thickness of the filter material placement will depend on the amount of seepage flow, with small seeps potentially requiring only fairly thin filter blankets, while large seepage flows may necessitate several feet of filter material covered by a berm of miscellaneous material. For large flows, initial placement of filter materials may simply get washed or eroded away due to concentrated flows. In these cases, it may be necessary to first place some fairly large (perhaps medium gravel to cobble size or larger) materials over the seep in order to disperse the flows before placing filter materials and subsequent cover layers. With the use of such a "diffuser," it is important to limit the areal extent and thickness of the initial coarse layer because eroding sediments in the seepage pathway will begin to fill the interstices of the coarse zone (and internal erosion will likely continue). Furthermore, it will be important to then construct a proper sequence of filtered layers of sufficient weight and thickness above the coarse zone to ensure an effective filtered exit. A surrounding shallow trench filled with filter materials may be necessary to intercept seepage that wants to escape around a filter placement.

8.5.4.3 Upstream Fill Placement

In a severe emergency where an internal erosion pathway has led to significant discharge of muddy seepage, dumping fill into the reservoir at the suspect seepage entrance point can be a means of stopping the flow. When the entrance point is reasonably well known (perhaps by the presence of a whirlpool), all possible efforts to seal the entrance should be taken. When Reclamation's A.V. Watkins Dam nearly failed from piping in 2006, seepage flows continued (to other exit points) even with the placement of downstream filter materials. It was not until material was placed in the reservoir and on the upstream face that most of the seepage ceased. The placement of downstream filters may slow or stop internal erosion but may do little to stop the seepage. Upstream fill placement can either seal the entrance point or introduce soils that are transported by the seepage flow through the developing void and, ultimately, plug against the downstream emergency filter. This method is most likely to be successful when used in combination with the placement of a downstream filter/berm.

As was done at Reclamation's Ochoco Dam, consideration should be given to including specific materials in the upstream fill that can be easily identified in later forensic investigations. Such additives may include aquarium rock or cinders. These materials can act as a marker deposit to assist in locating the internal erosion pathway upon excavation.

8.5.4.4 Grouting

Procuring a specialty grouting contractor may take some time, even in an urgent situation; thus, grouting is not a particularly common measure for many emergency situations. However, when there is reasonable certainty that the seepage-related failure mode is occurring slowly, grouting may be feasible as a temporary measure before a final corrective action is determined. In addition, there are means of gravity grouting that do not entail specialty work and can be performed on short notice. These types of remedial grouting have been used at notable internal erosion incidents as noted below.

A possible use of grouting might be to seal collapsed or improperly designed drains that have become an unfiltered exit for internal erosion. These drains might include toe drains or structure underdrains. The outlet works stilling basin underdrain system at Enders Dam is such an example. Once a sinkhole was discovered near the stilling basin, and video surveys indicated sediments in the underdrains, grouting of the drains was one of the temporary actions to mitigate the potential for internal erosion (and will be part of the permanent repair).

In addition, grouting might be used to fill voids that have been detected in an embankment or foundation. For example, when a large void was discovered in

the USACE's East Branch Dam, the sinkhole was filled by gravity grouting shortly after it was detected. At Mosul Dam in Iraq, which is founded on a soluble foundation that apparently continues to deteriorate, bedrock grouting is performed through galleries on almost a continual basis. At Reclamation's Horsetooth Dam, when a sinkhole was found above a sloping limestone layer exposed near the upstream toe of the dam, grout was tremmied into three drill holes that were drilled to intercept the void in the limestone.
There are potential limitations with grouting, including the following:

- Grout can be washed out if grouting is performed in areas subject to large seepage flows.

- Grouting may simply redirect seepage flows that could result in internal erosion in another area, enlarge existing voids, or create new ones.

- Grouting could lead to plugging of filters and drains that would otherwise help control seepage.

- Grout can deteriorate or leach away with time.

- Grouting carries the risk of hydraulically fracturing soils if not carefully performed [13].

- Traditional cement grouting may not be particularly effective in soils, and chemical grouting may need to be considered.

8.6 Seepage Monitoring

8.6.1 General

Both worldwide and Reclamation experience with embankment dams has shown that the primary risk of failure or incident will most likely be associated with internal erosion, particularly during first filling, and in older dams without filters. Hence, it is extremely important to carefully monitor seepage-related behavior at embankment dams. It has been demonstrated through case history experience that monitoring can detect changing conditions at a dam that can, in turn, lead to investigations or intervention that prevent a potential seepage issue from developing into a dam failure. Thus, the importance of a thorough seepage monitoring program cannot be overstated. The following paragraphs discuss important components of a comprehensive monitoring program.

8.6.2 Instrumentation

Instrumentation in embankment dams can serve a number of purposes, including: (1) verifying design assumptions and expectations, (2) facilitating a better understanding of embankment dam behavior, (3) monitoring actual performance at a site, (4) diagnosing anomalous behavior at a dam, and (5) predicting future behavior under potentially different loading conditions. Within the context of evaluating potential seepage issues at a dam, purposes (3) through (5) tend to be of primary interest. For most new or existing dams, it will be useful and important to have instrumentation to measure the effects of seepage, as instruments can provide quantitative data with which to more thoroughly analyze and evaluate seepage behavior, both under existing conditions and under potentially higher-than-experienced reservoir levels. Seepage-related behavior of most interest to dam safety involves seepage flows and water pressures, and there are several types of instrumentation that can provide useful information on these behaviors. The following sections do not discuss specific details of the various instruments. Available references for readers seeking more details on specific instrumentation include Chapter 11," Instrumentation," of this design standard; Reclamation's *Water Measurement Manual* [61] and *Embankment Dam Instrumentation Manual* [62]; American Society of Civil Engineers *Guidelines for Instrumentation and Measurements for Monitoring Dam Performance* [63]; and Dunnicliff's *Geotechnical Instrumentation for Monitoring Field Performance* [64].

8.6.2.1 Piezometers

Piezometers are widely used in embankments and their foundations to measure water levels or hydraulic pressures at various locations. A typical piezometer will have a specific influence zone within a limited interval or area, which enables the determination of hydraulic pressures or water levels at distinct locations within the soil or rock strata. This distinguishes it from an observation well, which is not screened in a specific zone. There are several different types of piezometers, which are described in the references listed above. Common piezometers at Reclamation dams include slotted-pipe piezometers, porous-tube piezometers, hydraulic (twin tube) piezometers, and vibrating-wire piezometers. Advantages and disadvantages of the various types of piezometers, as well as their applicable usage, are described well in the references.

As described earlier in this chapter (section 8.3.3.1), piezometer data can provide information on the location of the phreatic surface, indicate the location of seepage pathways, allow an estimate of hydraulic gradients, determine pore pressures that may impact stability, and identify changes in seepage behavior that may indicate developing internal erosion pathways. In addition, piezometric data can provide insights on the effectiveness of seepage control and seepage reduction features. Given the variety of potential benefits, the installation and inclusion of piezometers as a means of evaluating seepage concerns and monitoring performance is widely accepted as a prudent and value-added practice. However, there are a few cautions and comments regarding piezometers that are worthy of brief discussion.

Piezometer data should be carefully studied to ensure that behavior appears reasonable. Instruments can be flawed or develop problems later, which can result in erroneous data. Such problems may include clogging (perhaps by vandalism) or shearing (perhaps during installation) at some point in a standpipe, a faulty seal in an installation of multiple piezometers in a drill hole (leading to interconnection of influence zones and potentially inaccurate portrayal of water levels and pressures in individual zones), and deterioration of the instrumentation with time (as observed in many of Reclamation's hydraulic piezometer installations). In addition, it is critical that the analyst understands the influence zone of each piezometer, which will require review of the drill log. Improper location and installation of the sanded zone and/or screened interval of piezometers can mean that pressures and water levels may be influenced by more than just the one layer or zone.

When attempting to evaluate risks of internal erosion, it should be recognized that piezometers may need to be at just the right location (i.e., in the concentrated leakage path) to determine gradients or provide advance warning of a developing problem. Believing that seepage problems will be detected because a site has numerous piezometers is false confidence given the heterogeneity of most foundations and the potential variability of embankment soils, as wells as the vastness of an embankment or foundation relative to a piezometer influence zone.

When considering the potential locations for monitoring seepage behavior with piezometers, attention should be focused on areas that are expected to have more seepage, higher gradients, and the potential for flaws or defects. More pervious portions of foundations and abutments, perhaps due to a change in geology or material type, are likely to convey more seepage than other locations in the dam. Higher gradients may exist in the vicinity of cutoffs such as cutoff trenches, grout curtains, and cutoff walls; piezometers upstream and downstream of these cutoffs can allow for gradient calculations. Adverse benches or steep slopes in the foundation bedrock, as well as penetrating features such as outlet works conduits or spillway walls, can provide the potential for cracking in an embankment and potential higher seepage flows or pressures. When there is a confining layer in the foundation, piezometers at different depths in both the underlying pervious layer and overlying confining layer can greatly benefit the evaluation of uplift concerns.

Since the drilling for a piezometer installation in a dam core can cause hydraulic fracturing, extreme care should be given to the decision to install such a piezometer and to the installation and procedure for installing the instrument.

8.6.2.2 Observation Wells

Observation wells tend to be much less useful than piezometers for one primary reason. Observation wells are frequently screened over a large depth, while piezometers usually have specific influence zones within a limited interval of a drill hole. While piezometers measure pressures or water levels at distinct locations within the soil or rock strata, a water level in an observation well may be the result of seepage through only one particular permeable stratum within the drill hole, or it may be a composite water level resulting from different layers with different permeabilities. This is not to say that observation well data are not useful; rather, it is simply important to recognize the limitations or uncertainties associated with data from observation wells. In general, observation wells can provide a reasonably accurate determination of ground water levels in fairly homogeneous deposits or aquifers or a general idea of ground water levels in abutments and surrounding areas at a dam. However, for any seepage analysis or evaluation where pressures and water levels for specific layers are needed, data from observation wells will likely not be very useful.

8.6.2.3 Seepage Measurements

Seepage flows can be measured by a number of different means. For small seeps, the "bucket and stop watch" method is sometimes used. This method simply consists of monitoring the amount of time it takes to fill a specific volume container, and then converting to a flow value such as gallons per minute. In other applications, flowmeters are used to monitor seepage or drain flows. However, for most cases, weirs or flumes are typically used to measure flows because these devices are relatively inexpensive and quite reliable if properly installed and maintained. The *Water Measurement Manual* [61] contains detailed descriptions of the various types of weirs and flumes, as well as numerous tables to convert weir or flume staff gage readings to flow rates.

At embankment dams, it is typical to measure flows at all producing drains that are accessible. This generally includes toe drain outfalls, relief wells, horizontal drains and flows in drainage tunnels, and drains in appurtenant structures. Since an embankment toe drain can be several thousand feet long, it is good practice to include several inspection wells along the toe drain system with the capability to measure flows at each [57]. Flumes or weirs, either prefabricated or cast in the bottom of these wells, can provide separate measurements of seepage flow along the length of the toe drain, thereby helping to determine which parts of the embankment or foundation have the most seepage.

Seepage through a dam and its foundation is rarely completely intercepted by the designed internal drainage systems. Some seepage passes through the foundation and is not visible in the area of the dam, and, thus, is not measured. At other locations, unanticipated surface seeps occur. If the surface seepage is of sufficient flow to be measurable and can be channelized, the flows from these seeps should be routinely monitored.

There is typically more certainty with the data from seepage measurements than with piezometric data, given the relative simplicity of the measurement devices. However, it is important to ensure that seepage is not bypassing weirs and flumes, staff gages are properly located and maintained, and the pathway into and out of the measurement device is free of weeds, sediments, and debris. Equally important is that seepage measurements are taken in generally dry weather periods, so that the influence of recent precipitation and surface runoff or infiltration does not bias the seepage readings. The schedule for periodic monitoring (L-23) for a dam typically states that seepage readings (and routine visual monitoring) should be carried out when no rainfall or snowmelt has occurred in the previous 24 (or perhaps 48) hours.

8.6.2.4 Turbidity or Sediment Collection

For the evaluation of internal erosion failure modes, the presence of any transported soil particles at seepage measurement points is of critical interest. Evidence of material transport by seepage flow is direct evidence of initiation/progression of a seepage-related failure mode. Increases in seepage flow rates (corrected for changing reservoir levels) are an indirect means of such detection because other reasons for seepage flow rate increases may exist (such as degradation of a grout curtain) that do relate to erosion and transportation of materials along a seepage flow path.

In most instances, sediment transport is monitored by including stilling areas or sediment traps upstream of a weir or flume, thus providing a still water area in which soil particles can drop out of seepage flows. The stilling area should be lined so that sediment can be easily discerned. It is important to note the presence of any sediment in these stilling areas when flow measurements are taken and to periodically clean out the sediment traps. The volume of the sediment should be measured and recorded, so that trends in the sedimentation rate can be ascertained. In addition, care should be taken to prevent adjacent soils from being deposited in the collection area, perhaps by wind action or surface runoff. Protective covers are suggested to isolate weirs and flumes from soil contamination. Given the utmost importance of monitoring for soil transport (internal erosion), all seepage flow measurement locations should be carefully designed to include some provision for stilling flows and allowing sediments to drop out.

Another method of monitoring sediment transport is the use of turbidity monitoring units. However, the use of turbidity monitoring units is not recommended. Reclamation experience with these devices indicates that they tend to require relatively extensive care and maintenance and, in most cases, do not provide reliable information relative to possible sediment transport by seepage flow. In addition, the devices do not measure the total accumulation of sediments.

A means to monitor sediment transport from drains is to direct it through a geotextile. The geotextile can be observed and weighed periodically to confirm clear flow or the presence (and amount) of sediment. Drains at Navajo Dam and weepholes at Kachess Dam have used such installations.

8.6.2.5 Thermal Monitoring

Thermal monitoring is a means of detecting the presence of seepage by temperature measurements, typically in soil foundations. It is not a quantitative measurement of flow but, rather, an indicator of seepage concentrations and flow paths. In addition, it can indicate changing seepage conditions, which would be of value in monitoring for potential internal erosion failure modes. Reclamation has rather limited experience with this instrumentation; however, it is more widely used in Europe. Temperature measurement devices can be coupled with fiber-optic cables to essentially produce a long sensor that can be used in a vertical or horizontal manner. Thus, for dams of significant length (or for levees) where this type of installation could be installed along a line at the toe, thermal monitoring may be of particular interest as a means of identifying seepage concentrations that can be monitored more closely with additional conventional instrumentation.

8.6.3 Visual Observations

8.6.3.1 Routine Inspections

Frequent routine inspections of a dam and its surroundings are a critical part of a dam safety monitoring program. Comprehensive visual observations are key to detecting ongoing internal erosion and seepage-related problems before they progress to dam failure. At Reclamation facilities where incidents of developing seepage or internal erosion have been reported, most have been detected by visual observation as opposed to unusual instrumentation readings. While instruments tend to be focused in very limited areas of influence, a comprehensive visual inspection will cover the entire dam and surrounding area, making it more likely that changing conditions or newly developing seeps will be detected.

The frequency of the inspections should be tied to the risks of the potential failure modes. Thus, dams above large population centers and dams with a history of seepage issues will likely require more frequent inspection. Most Reclamation dams require a documented inspection at least monthly, using an Ongoing Visual Inspection Checklist (OVIC), which is updated as needed and, at other times, as appropriate, based on changing circumstances at the dam site. Reclamation facilities typically get visited by operations personnel weekly or several times a week during irrigation season when the reservoir is high. These operational personnel thus provide an additional informal visual observation more frequently than monthly during important times of reservoir operation. In addition, the OVIC usually stipulates more frequent inspections after an earthquake, during a flood or unusual spillway operations, or whenever the reservoir exceeds previous historic maximum levels.

In addition to requiring an overall inspection of the dam and downstream areas, the OVIC will typically identify specific areas to receive special, close attention. Obviously, any areas of existing seepage are critical and should be watched closely. Case histories suggest that areas adjacent to conduits or penetrating structures within an embankment are particularly vulnerable spots for seepage problems. Locations of dramatic topographic changes are also suspect due to the possibility of cracking.

8.6.3.2 Closed Circuit Television Examinations

Closed circuit television (CCTV) surveys have proved quite useful for inspection of drainage pipes such as those included in toe drains and structure underdrains. CCTV surveys are also useful at dams with small outlet works pipes, such as the corrugated metal pipe conduits frequently found at Bureau of Indian Affairs dams. As pipes age, they can deteriorate, and the result is an increased potential for providing an open or unfiltered exit for seepage. CCTV surveys using small cameras mounted on self-propelled systems or pushed by rods or cables can provide views of the pipe interior and signs of collapse, corrosion, joint separation, root infestation, or sediment deposition. When evaluating potential seepage-related failure modes, these surveys can provide important insights into whether internal erosion has initiated and whether the drains are serving as an appropriately filtered exit [65] or if the filtered drainage system has been compromised at one or more locations.

8.6.4 Integrated Monitoring Program

Since seepage issues can potentially lead to catastrophic failure of an embankment dam under certain conditions, it is critical to integrate seepage monitoring with an overall assessment of the safety of the dam. The following paragraphs highlight some important considerations for integrating seepage monitoring with other components of Reclamation’s dam safety activities.

8.6.4.1 Combination of Instrumented and Visual Monitoring

Both instrumented readings and visual observations have merits and limitations. For maximum effectiveness in observing seepage behavior, an effective monitoring program should emphasize a combination of instrumented and visual monitoring. Reclamation creates a Schedule for Periodic Monitoring, or L-23 form, for each dam in their program. The L-23 lists all instruments to be read, as well as the frequency for readings. The L-23 also specifies the frequency of visual inspections using the dam’s OVIC.

8.6.4.2 Portrayal and Review of Data

At Reclamation dams, instrumentation data are typically collected by either irrigation district personnel or Reclamation staff. The data are checked at the time the readings are obtained in the field to verify that readings are reasonable. Data are then provided to the Instrumentation and Inspections Group (86-68360), who enter the data (as applicable) and perform a check to determine if the readings appear valid and within expected limits or are anomalous. Anomalous readings typically generate a contact with the field to verify results; if verified, additional discussions with appropriate technical staff in Denver are initiated. To help define anomalous behavior, each instrument typically has "performance parameters," which essentially define the range of expected behavior for given conditions (such as reservoir level). By comparing the observed performance against these parameters, anomalous readings are fairly obvious. Instrumentation data are stored, plotted, and available for review and evaluation through the Data Acquisition and Management System (DAMS). With this database, engineers can log in and generate a variety of reports and graphs that portray instrumentation readings at any given Reclamation dam. Geotechnical engineers responsible for a particular embankment dam are strongly encouraged to periodically access the DAMS database and review current instrumentation (and visual observation data). Viewing plots of instrumented behavior over time can help indicate whether the instruments are showing consistent behavior over time. In addition, "scatter" plots for seepage and piezometer readings versus reservoir elevation can demonstrate whether seepage and pressures respond to reservoir levels, and whether the trends are changing. Scatter plots may well be the most important plots to study when looking for trends in seepage or piezometric behavior. Example scatter plots for seepage and piezometric data are shown in figures 8.6.4.2-1 and 8.6.4.2-2.

Instrumentation data, by itself, is useful in observing trends and comparing behavior to past performance. However, a much more thorough understanding of the data can be achieved by developing embankment cross sections depicting the dam features, foundation geology, and all the instrumentation locations. Adding instrument plots to these base cross sections enhances understanding of how the overall site conditions may influence or explain the performance data.

8.6.4.3 Combination of Monitoring and Failure Mode Evaluation

A monitoring program including visual inspection and instrumentation is a critical component of a dam safety program. The other essential component is an engineering evaluation of the potential failure modes for a given dam. By looking thoroughly into the potential weaknesses and vulnerabilities of each dam, a better quality monitoring program will result because inspectors will know what to look for. In the Reclamation dam safety program, the failure mode evaluation is done on a regular cycle as a part of the comprehensive review. The first step of this evaluation of failure modes is to develop a thorough list of potential failure mechanisms. The next step is to assign factors that make each failure mode likely or unlikely. The final step in Reclamation's process is to use event trees to lay out each step of the failure process and estimate an annual probability of failure. A useful reference for a better understanding of the development of failure modes and the evaluation risk process is the *Best Practices in Dam and Levee Safety Risk Analysis* training manual [7].

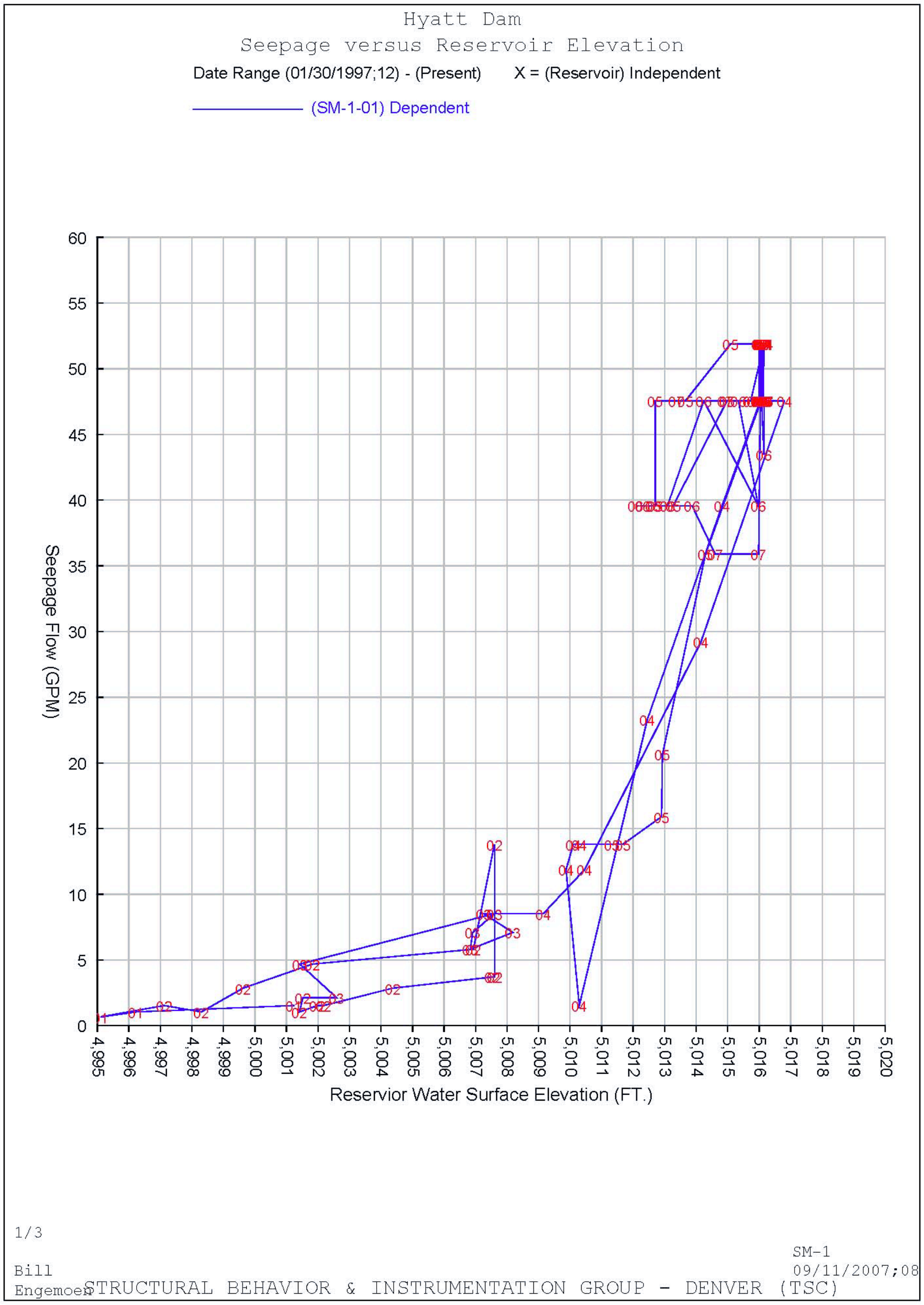


Figure 8.6.4.2-1. Scatter plot for seepage data.

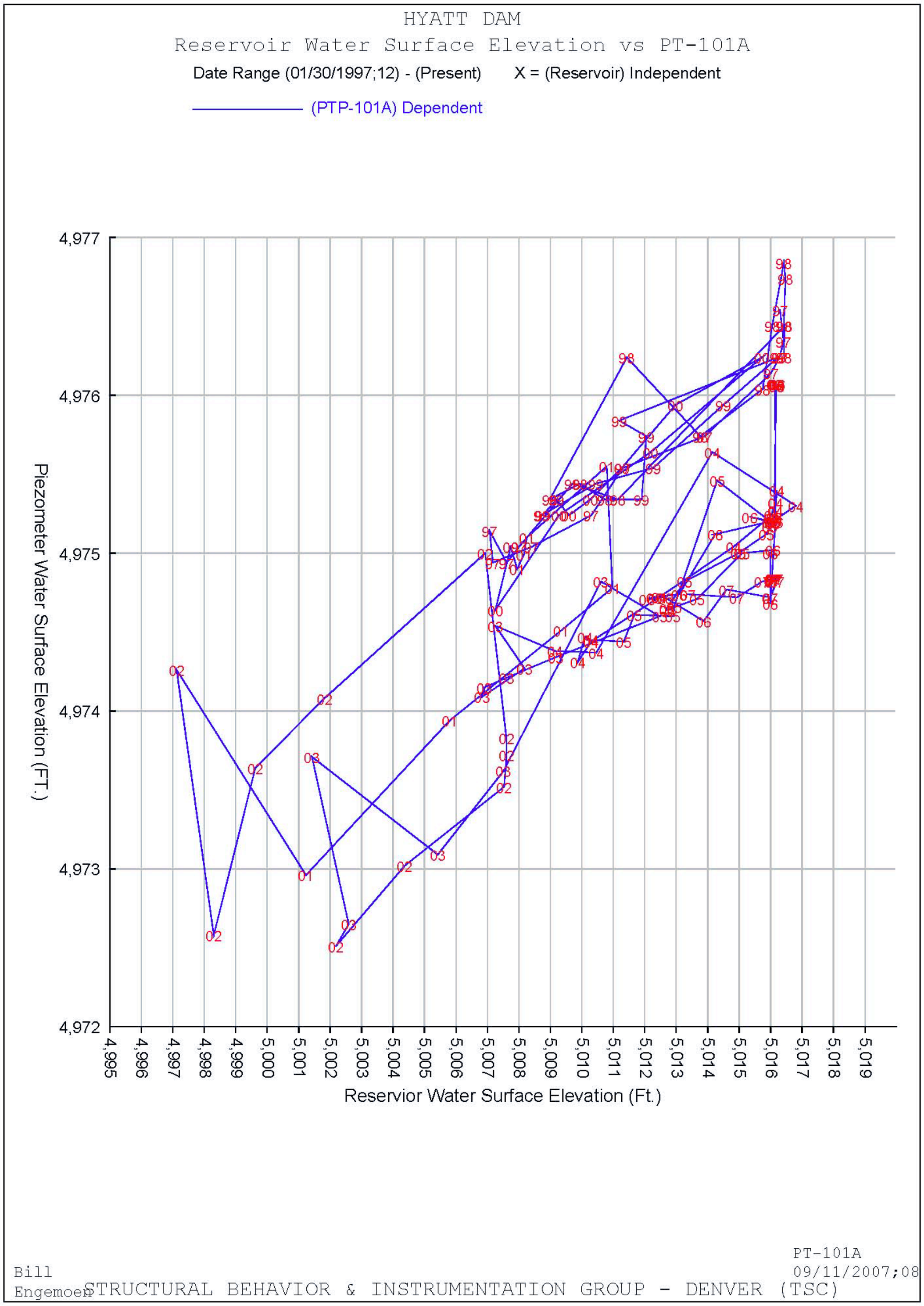


Figure 8.6.4.2-2. Scatter plot for piezometric data.

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Appendix A

Considerations for Uplift Pressures and Exit Gradients at the Downstream Toe of Embankments

Appendix A

Considerations for Uplift Pressures and Exit Gradients at the Downstream Toe of Embankments

General

Section 8.2.2.3 presents a discussion of uplift computations using the "total stress" and "effective stress" methods. This appendix examines an example situation and compares the computation of uplift pressures using the two methods.

Uplift of a Confining Soil Layer – Example Calculations

To help illustrate the factors involved in uplift calculations, consider the situation at the downstream toe of an embankment without a positive foundation cutoff as shown in figure 1.

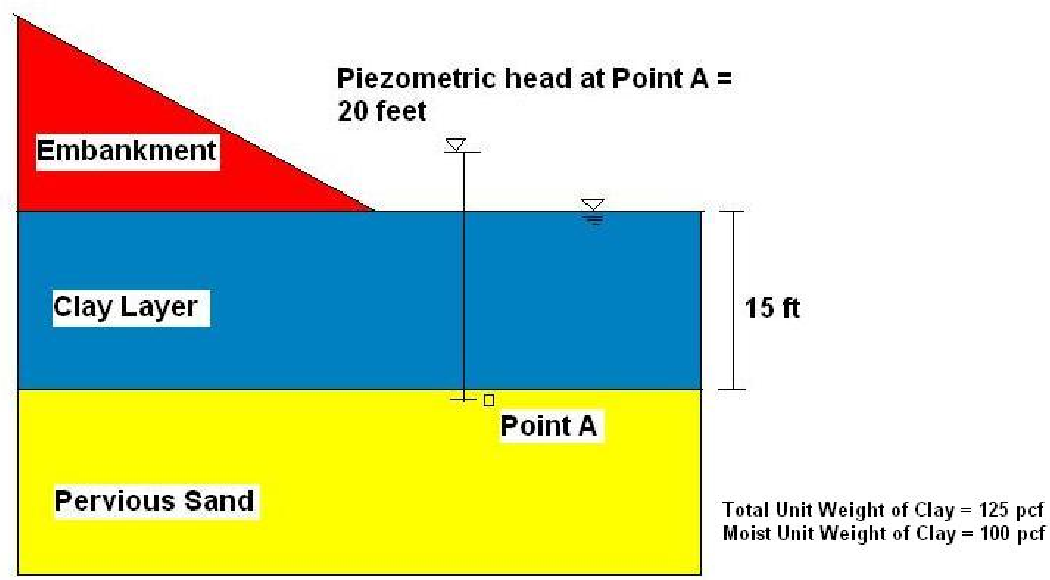


Figure 1. - Confined Layer at Downstream Toe of Embankment

The upper confining layer of the foundation consists of a 15-foot-thick layer of clay overlying a thick deposit of pervious sands. Piezometers in the pervious sand layer indicate that high pressures exist in this foundation layer. In this example, a piezometer at the top of the sand layer has a reading of 20 feet of pressure head, or 5 feet above the ground surface at the toe. As is done with most classical representations, the clay layer is assumed to be saturated (i.e., the phreatic line is at the top of the existing ground). Considering total forces, the uplift force at

Point A (assumed to be at the top of the pervious sand layer) will be the pressure head in feet multiplied by the unit weight of water, or (20 feet) x (62.4 pounds per cubic foot [lb/ft^3]), which equals 1,248 pounds per square foot (lb/ft^2). The total resisting force is the saturated unit weight of the clay multiplied by the thickness of the layer, or (125 lb/ft^3) x (15 feet), which equals 1,875 lb/ft^2. Note that the effective stress at Point A is the difference of these two pressures, or 1,875 – 1,248, or 627 lb/ft^2. Using total forces, the factor of safety could be defined as the total weight of the clay layer (per unit area) divided by the uplift pressure (per unit area). This equates to 1,875/1,248, or a safety factor of 1.5.

A different approach to calculating forces would be to consider buoyant forces and seepage forces. (Note that both this approach and the preceding method are described in chapter 17 of Lambe and Whitman, 1969) [4]. With the consideration of seepage forces, the uplift pressure is essentially the differential piezometric head at the top of the pervious sand layer. In figure 1, this would equate to 20 feet (confined pressure at top of sand layer) minus 15 feet (hydrostatic pressure at the base of the clay layer), or 5 feet. The uplift force is therefore (5 feet) x (62.4 lb/ft^3), or 312 lb/ft^2. The resisting force is the buoyant weight of the clay layer multiplied by its thickness. This equates to (125 – 62.4 lb/ft^3) times 15 feet, or 939 lb/ft^2. Note that the effective stress at Point A calculated by this approach is the difference in the two pressures, 939 minus 312, or 627 – the same answer as computed above. However, if one were to compute the factor of safety between the two pressures, 939 lb/ft^2 divided by 312 lb/ft^2, the value would be 3.0, or twice the safety factor computed above.

Obviously, the condition at Point A is the same regardless of which approach is used to compute pressures and forces. In this example, the clay layer has sufficient weight and thickness to overcome the high water pressures in the underlying sand layer and is, thus, safe from uplift under the assumed conditions. Whether the factor of safety against uplift is 1.5 or 3.0 depends solely on the selection of the method of analysis; in other words, it depends on how one chooses to define the safety factor. In an attempt to see the differences in the two methods, a few variations of the example will be considered.

For instance, often in real situations, there are no piezometers in the overlying clay layer, since engineers and geologists might be more concerned with the known pervious layer. However, the seepage gradient and pore pressures in the clay layer can make a significant difference to an evaluation of uplift. For an embankment that has a fluctuating reservoir level (or a levee that has yet to experience an appreciable river stage) and a quite impermeable clay layer, it is possible that the clay layer is unsaturated or partially saturated, even after many years of operation. In this case, there is relatively little pore pressure in the clay, and the resisting force of the confining overburden can be represented by the in-place unit weight of the clay times its thickness. The clay in figure 1 is assumed to have a moist (or in-place) unit weight of 100 lb/ft^3. Therefore, the total resisting force (per unit area) of the clay layer in this case is (100 lb/ft^3) x

(15 feet), or 1,500 lb/ft^2. The uplift force at the base of the clay layer remains the same, at 1,248 lb/ft^2. The factor of safety for this condition is therefore 1,500/1,248, or 1.2. This is less than the safety factor of 1.5, which is reasonable as the clay layer has a lower unit weight because it is not saturated.

Since this condition assumes that the clay layer is not saturated, it thus has no seepage force flowing upward through it. Hence, the buoyant weight and seepage force method does not really apply. The resisting force is simply the moist unit weight of the clay minus zero pore pressure, or 1,500 lb/ft^2. The uplift force is the total head measured at Point A, since there is no water level in the clay above. The uplift is therefore 1,248 lb/ft^2, resulting in the same safety factor as previously calculated. This is a significant reduction in the safety factor, from 3.0 to 1.2 – a much larger reduction in apparent stability than demonstrated by the total force approach.

Considering this example further, suppose the piezometric head in the clay layer is assumed to be at the midpoint of the layer. In other words, the phreatic surface in the clay is at a depth 7.5 feet below the ground surface (and, thus, the hydrostatic head in the base of the clay layer is 7.5 feet). From a total forces standpoint, the uplift pressure is still the 20 feet of head at the top of the sand layer multiplied by the unit weight of water, or 1,248 lb/ft^2. The resisting force is now the sum of a portion of saturated clay and the upper portion of "moist" clay. This equates to (7.5 feet) x (125 lb/ft^2) plus (7.5 feet) x (100 lb/ft^2), or 1,687.5 lb/ft^2. The factor of safety against uplift is now 1,687.5/1,248, or 1.35. This value is between the previous calculations of 1.5 for fully saturated clay and 1.2 for moist clay.

Using the seepage force/buoyant weight approach, the net uplift seepage force is 20 minus 7.5 feet, or 12.5 feet times the unit weight of water, which equals 780 lb/ft^2. The resisting force is the moist weight of the soil times 7.5 feet (125 x 7.5 = 937.5 lb/ft^2) plus the buoyant weight of the soil times 7.5 feet (62.6 x 7.5 = 469.5 lb/ft^2), for a sum of 1,407 lb/ft^2. The resulting safety factor is 1,407/780, or 1.8; a value that is between the saturated and moist clay safety factors of 3.0 and 1.2, respectively.

Finally, to compare the differences between the two methods in a different way, one can compute the thickness of the clay that would result in a safety factor of unity for the original example, assuming the same 20 feet of pressure head at the top of the sand layer. With the total force method, for a safety factor of unity:

> Total unit weight of clay times clay thickness = Total uplift force
> (γ_t lb/ft^3) times (t feet) = (62.4 lb/ft^3) times (20 feet)
> t = (62.4)(20)/(125)
> t = 10 feet

With the buoyant force/seepage force method, for a safety factor of unity:

> Buoyant unit weight of clay times clay thickness = Net uplift force
> (γ_b lb/ft^3) times (t feet) = (62.4 lb/ft^3) times (20 – t feet)
> (62.6)t = (62.4)(20) – (62.4)t
> 125t = (62.4)(20)
> t = 10 feet

Thus, each method computes the same critical thickness of the clay layer that would result in a factor of safety of 1.0.

In summary, calculating uplift forces using a total force method and a buoyant weight/seepage force method results in the same computed effective stress at the base of a confining layer. Similarly, both methods compute the same thickness of the clay layer required to offset a specified uplift pressure. Based on the examples, the buoyant weight/seepage force safety factor appears more volatile (factor of safety changes dramatically). That method also appears to indicate safety factors (FS = 3.0 for original example) that generally appear somewhat higher than one would expect from most geotechnical engineering analysis cases. In other words, a safety factor of 3.0 would suggest extreme stability in an analysis of static stability, whereas the portrayed example of a blowout situation does not appear nearly so obviously stable. For this reason, the use of total forces to evaluate uplift safety factors is recommended.

Uplift Pressures Leading to Dam Failure

If calculations, such as illustrated above, indicate the potential for seepage gradients to approach the critical gradient or for uplift pressures to be near the resisting overburden pressures, it is possible that the embankment and foundation may experience sand boils (in a cohesionless foundation) or perhaps cracking of the low permeability confining layer. The failure mechanism that may pose the greatest risk to the dam is that these events will then lead to progressive backward erosion and ultimate dam failure. However, although potentially serious and worthy of immediate study and possible action, these conditions do not guarantee that the dam is in danger of failing. A followup step is to evaluate potential failure modes to determine the severity of the sand boils or uplift condition. Typically, this is done within Reclamation by conducting a quantitative risk analysis.

Describing the details of such a risk analysis is outside the scope of this document, but a few general considerations and concepts are worth a brief discussion. Following are some key factors that should be considered in determining the criticality of heave or uplift conditions at the downstream toe of an embankment dam.

Horizontal Hydraulic Gradient

Once the initial particle movement occurs at the exit point, backward erosion needs to initiate and progress in order to potentially lead to a serious concern. Thus, the horizontal hydraulic gradient (as opposed to the vertical gradient which was of initial concern) is a key factor that influences whether backward erosion will initiate or continue. As discussed in the standard, some laboratory models have shown that some erosion can initiate at low gradients on the order of 0.08 or less in clean, fine sands; and Lane's weighted creep method would suggest that erosion could initiate at a gradient of greater than 0.04 in very fine sands or silts. Furthermore, the Wister Dam experience suggests that internal erosion might occur in dispersive clays under gradients as low as 0.02. However, it is likely that higher gradients are needed for most conditions generally representative of foundations beneath embankment dams. Risk teams must carefully evaluate the potential gradients and the properties of the foundation soils to determine the likelihood that backward erosion will both initiate and continue.

Ability to Self-Heal

Even if backward erosion does initiate, there are factors that may cause the erosion process to stop, or self-heal. For one thing, erosion will not continue unless a "roof" can form in the developing "pipe." This usually requires an overlying material with some cohesion. If the embankment and foundation are comprised solely of cohesionless materials, there is a reasonable chance that backward erosion will not develop. Similarly, a heterogeneous foundation may present zones of different materials that may not be able to support a roof or may provide crackstopping or plugging materials for a developing pipe. These types of considerations are important discussion topics in a risk analysis to determine the potential for backward erosion to continue upstream to a sufficient point that could lead to a dam failure.

Erodibility of Soils

Another factor that plays a significant role in the progression of backward erosion is the relative erodibility of the foundation soils. This, in turn, depends on soil density and the amount of cohesion or induration (in conjunction with the hydraulic gradient). Thus, a good understanding of the foundation soil properties is critical to understanding the potential for erosion progression.

Other Internal Erosion Factors

There are several other factors that are considered in estimating the probability of dam failure resulting from internal erosion, which will not be discussed. However, additional considerations include the probabilities that the seepage exit is unfiltered, that the seepage flows are limited by some means, that the developing erosion can be detected and mitigated, and that the embankment is capable of breaching.

Embankment Slope Stability

In addition to an internal erosion, or piping, failure as discussed above, the high pore pressures in the foundation may also have a destabilizing effect on embankment stability due to the lowering of effective stresses. Thus, it is also important that slope stability be analyzed in addition to internal erosion potential. This can be done by using computerized approaches such as SLOPE/W and inputting the measured or projected values of foundation pore pressures, material strengths, and embankment and foundation geometry. Often, several assumed variations of pore pressures are evaluated in order to understand the sensitivity of the results.

Appendix B

Analysis and Design Equations and Charts

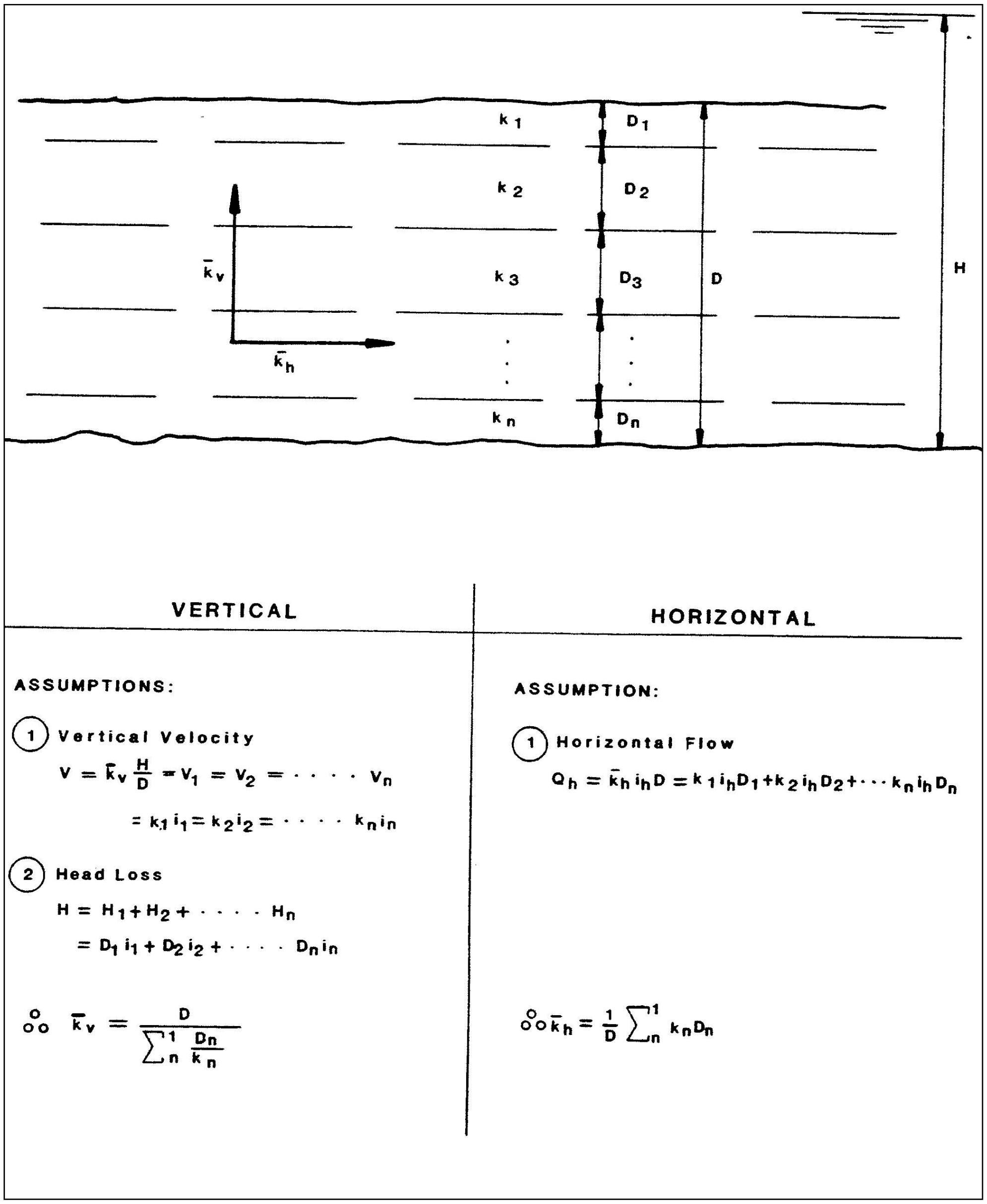


Figure B-1. Equivalent permeability of a stratified deposit.

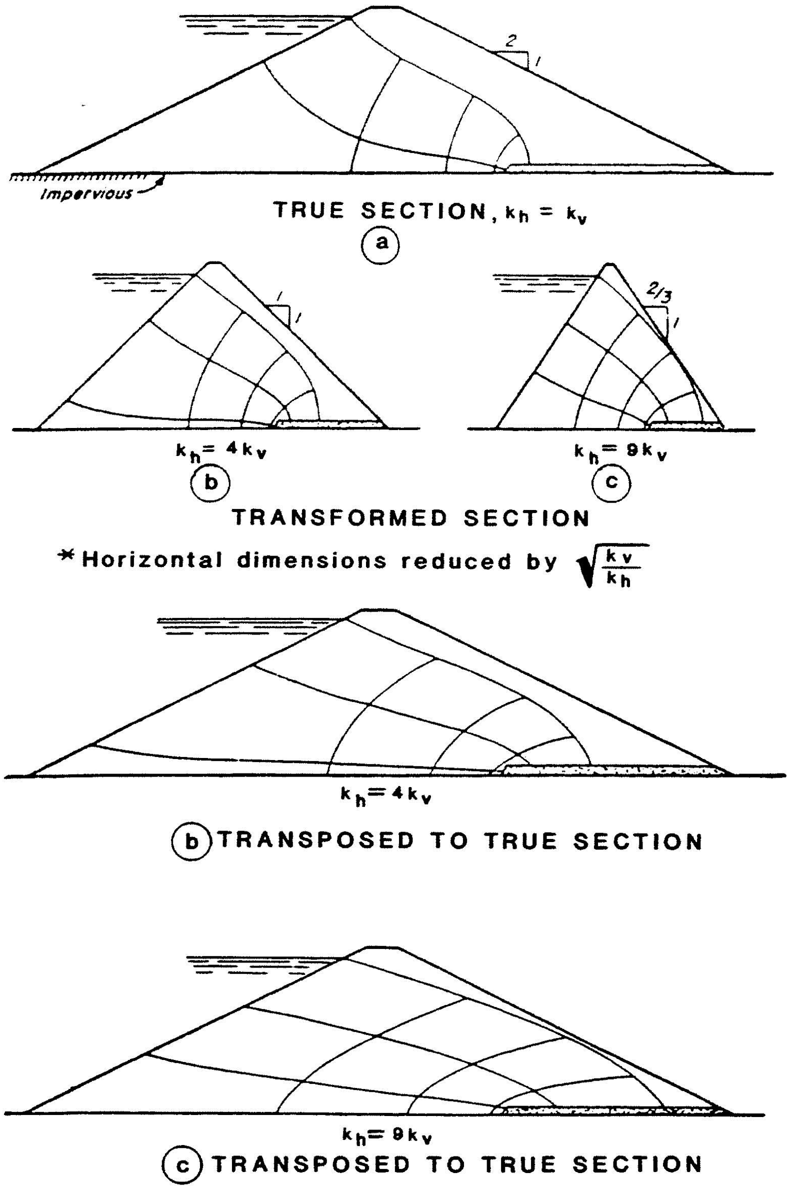


FIGURE B-2 – TRANSFORMATION METHOD

Figure B-2. Transformation method for analysis of anisotropic embankments [17].

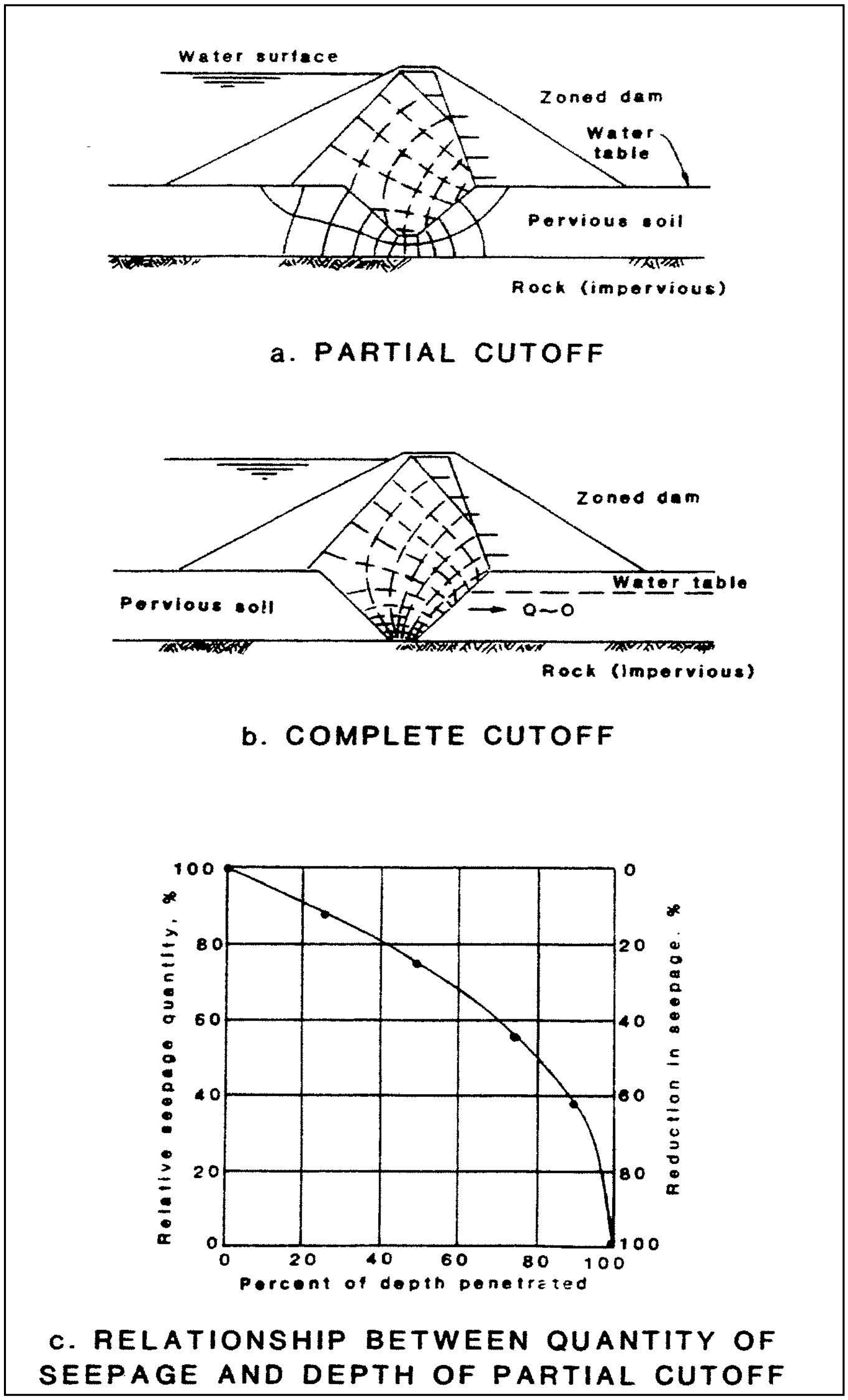


Figure B-3. Effect of partial penetration of cutoff trench [3].

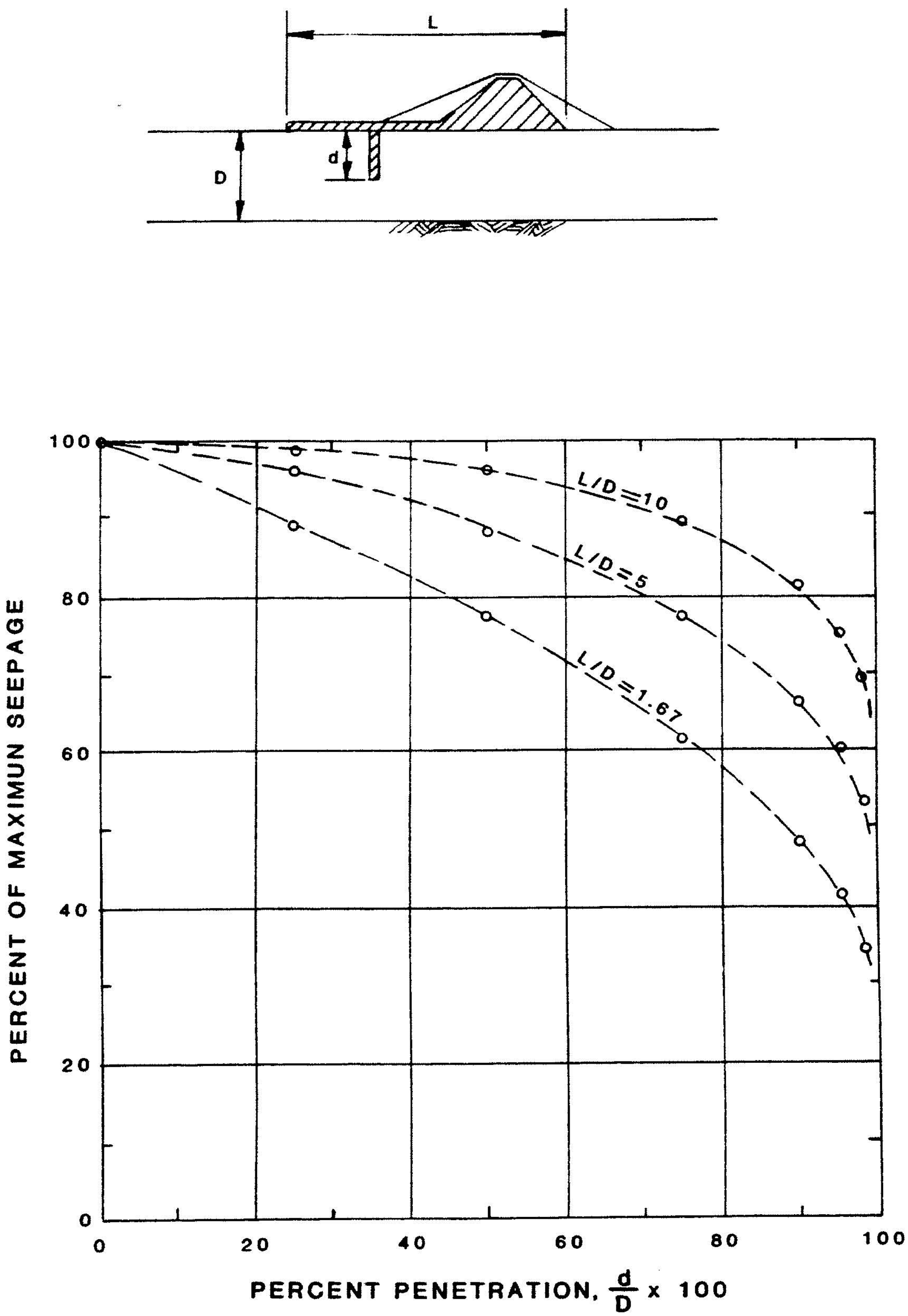


Figure B-4. Effect of partial penetration of cutoff wall [66].

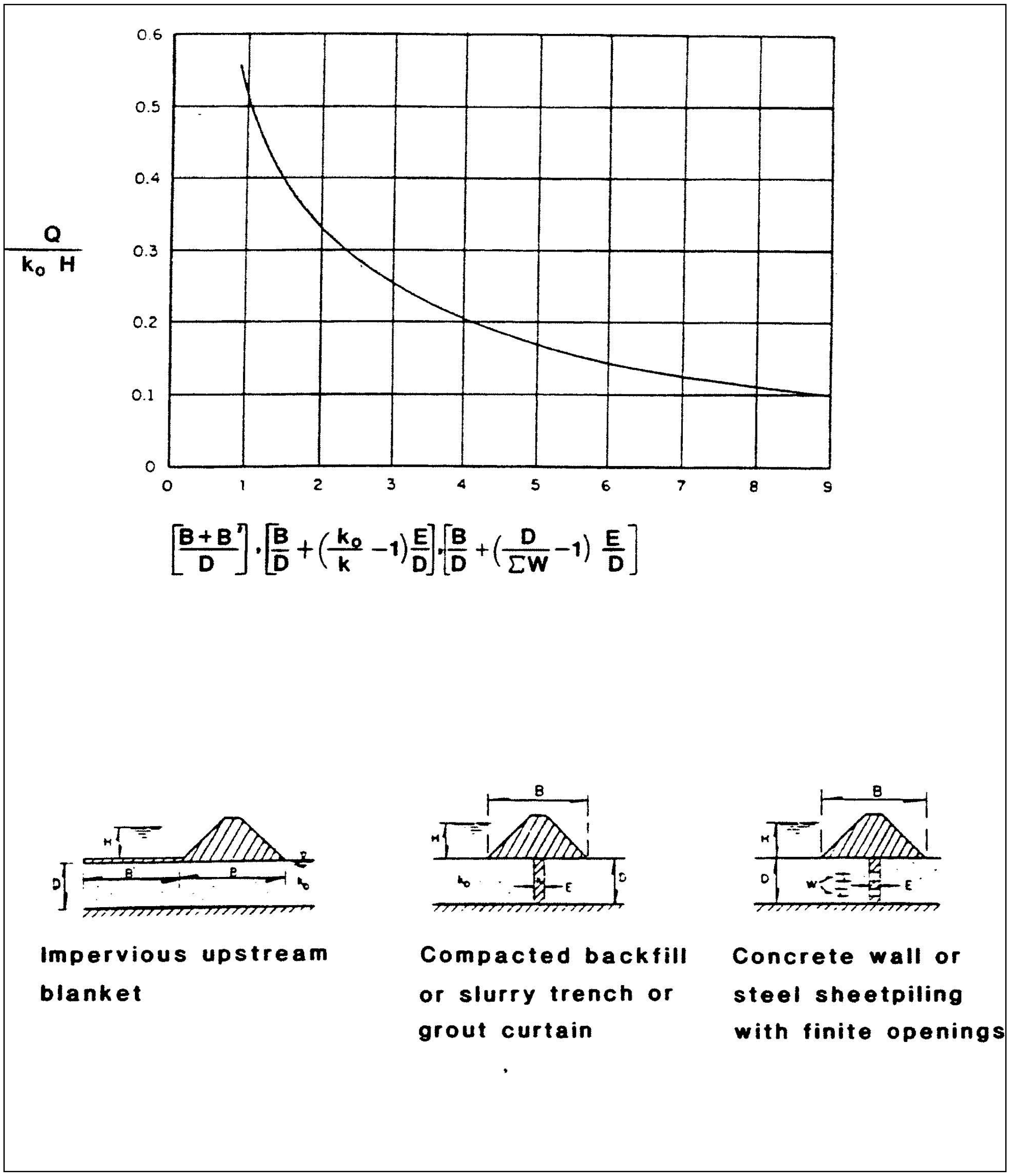


Figure B-5. Effect of partial penetration of cutoff wall [67].

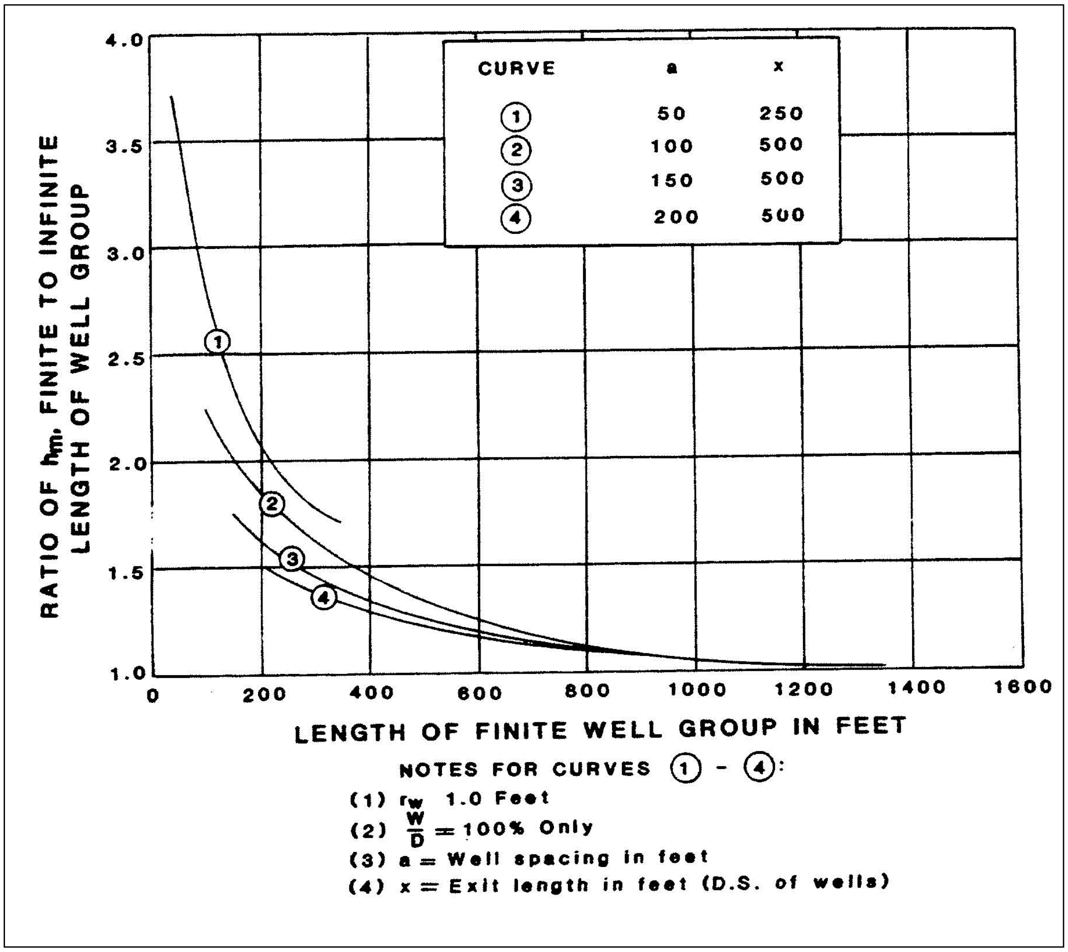
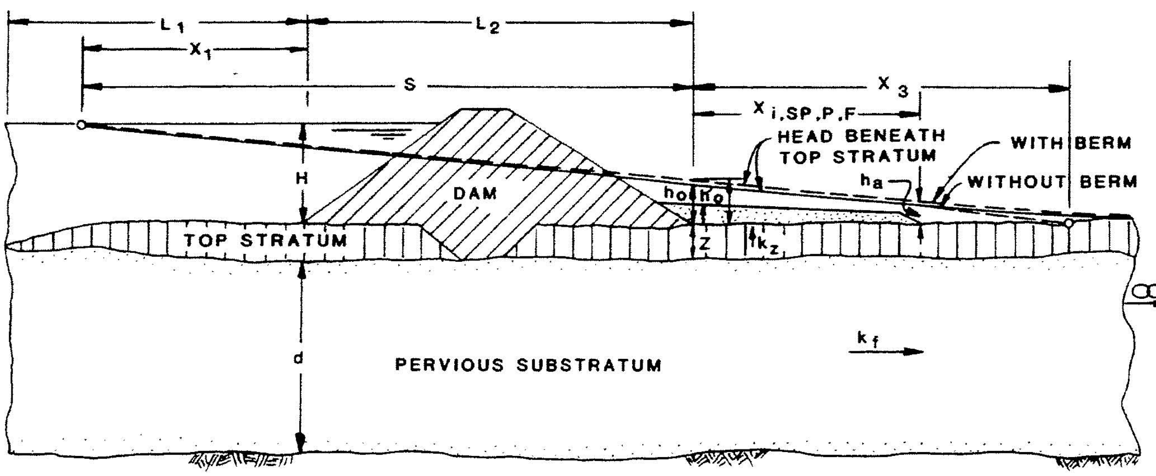


Figure B-6. Analysis of relief well systems [58].



$c = \sqrt{\dfrac{k_z}{k_f Z d}}$

γ'_z = Submerged unit weight of top stratum

γ'_t = Submerged unit weight of berm

γ_w = Unit weight of water

x_3 = Effective length of downstream blanket $= \dfrac{1}{c}$

x_1 = Effective length of upstream blanket $= \dfrac{\tanh cL_1}{c}$

h_o = Head at downstream toe of dam without berm $= \dfrac{HX_3}{S+X_3}$

h_o' = Head at downstream toe of dam with berm

i_o = Allowable upward gradient at downstream toe of dam (use $i_o = .2$)

i_a = Allowable upward gradient at toe of berm (use $i_a = .3$)

h_a = Allowable head at toe of berm $= i_a z$

$X_{i,SP,P,F}$ Required berm width

t = Required thickness of berm at toe of dam

k_t = Vertical permeability of berm

F = Factor of safety against uplift at toe of dam (use $F = 2.0$)

Q_B = Flow into berm per unit length of dam

IMPERVIOUS BERM ($k_t = 0$)

$$X_I = x_3\left(\frac{H}{h_a} - 1\right) - S$$

$$h_o' = H\left(\frac{X_3 + X_I}{S + X_3 + X_I}\right)$$

$$t = \frac{h_o' - Z\left(\dfrac{\gamma'_z}{F\gamma_w}\right)}{1 + \dfrac{\gamma'_t}{F\gamma_w}}$$

SEMIPERVIOUS BERM ($k_t = k_z$)

$$X_{SP} = \frac{-A + \sqrt{A^2 - 24(2+r)\left(1 + SC - \dfrac{H}{h_a}\right)}}{2c(2+r)}$$

WHERE:

$$A = 6 + 3SC(r + 1)$$

$$r = \frac{i_o}{i_a}$$

$$h_o' = h_a\left[1 + CX_{SP}\left(\frac{2+r}{6}\right)(CX_{SP})^2\right]$$

$$t = \frac{h_o' - i_o Z}{1 + i_o}$$

PERVIOUS BERM ($k_t > k_z$)

$$X_P = \frac{1}{3}(X_P + 2X_{SP})$$

$$h_o' = h_a\left[1 + CX_P + \left(\frac{2+r}{6}\right)(CX_P)^2\right]$$

$$t = \frac{h_o' - Z\left(\dfrac{\gamma'_z}{F\gamma_w}\right)}{1 + \dfrac{\gamma'_t}{\gamma_w}}$$

FREE DRAINING BERM

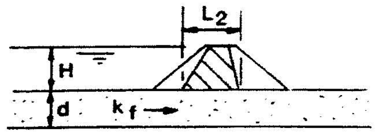
$$X_F = x_3 \log_e\left(\frac{h_o'}{h_a}\right)$$

$$h_o' = h_o = \frac{Hx_3}{S + x_3}$$

$$t = \frac{h_o' - Z\left(\dfrac{\gamma'_z}{F\gamma_w}\right)}{1 + \dfrac{\gamma'_t}{F\gamma_w}}$$

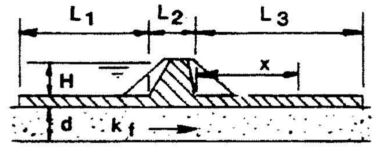
$$Q_B = \frac{k_f dH}{S + X_3}\left(1 - e^{-\frac{X_F}{X_3}}\right)$$

Figure B-7. Design of downstream seepage berm [39].



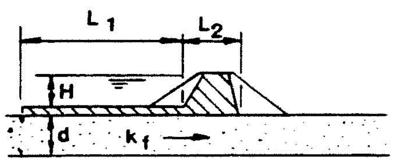
$$\$ = \frac{d}{L_2+0.86d}$$

$$h_x = 0$$



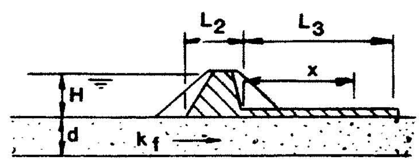
$$\$ \quad \frac{d}{L_1+L_2+L_3}$$

$$h_x = H\left(\frac{L_3-x}{L_1+L_2+L_3}\right)$$



$$\$ = \frac{d}{L_1+L_2+0.43d}$$

$$h_x = 0$$



$$\$ = \frac{d}{0.43d + L_2+L_3}$$

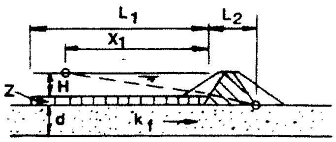
$$h_x = H\left(\frac{L_3-X}{0.43d + L_2+L_3}\right)$$

Head beneath blanket at distance x from downstream toe . . . . . . h_x

Shape factor for use in seepage equation . . . . $\$$

Seepage per unit length of dam . . . . $Q = \$ k_f H$

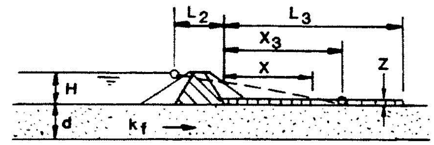
Figure B-8. Equations for impervious blanket computations [39].



$$x_1 = \frac{\tanh cL_1}{c}$$

$$\$ = \frac{d}{x_1 + L_2 + 0.43d}$$

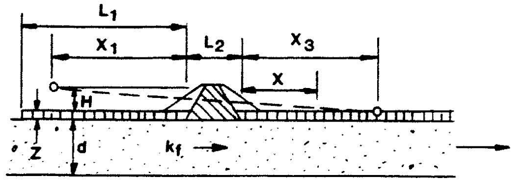
$$h_x = 0$$



$$x_3 = \frac{\tanh cL_3}{c}$$

$$\$ = \frac{d}{0.43d + L_2 + x_3}$$

$$h_x = H\left(\frac{x_3 - x}{0.43d + L_2 + x_3}\right)$$

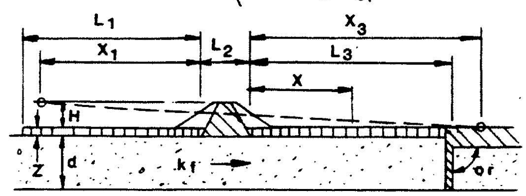


$$x_1 = \frac{\tanh cL_1}{c}$$

$$x_3 = \frac{1}{c}$$

$$\$ = \frac{d}{x_1 + L_2 + x_3}$$

$$h_x = H\,\frac{e^{-cx} x_3}{x_1 + L_2 + x_3}$$

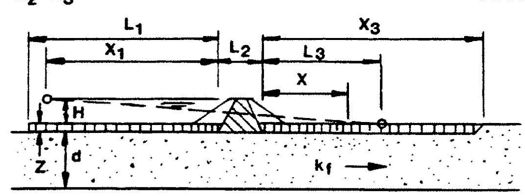


$$x_1 = \frac{\tanh cL_1}{c}$$

$$x_3 = \frac{1}{c \tanh cL_3}$$

$$\$ = \frac{d}{x_1 + L_2 + x_3}$$

$$h_x = H\left(\frac{\cosh c(L_3 - x)}{\cosh cL_3}\right)\left(\frac{x_3}{x_1 + L_2 + x_3}\right)$$



$$x_1 = \frac{\tanh cL_1}{c}$$

$$x_3 = \frac{\tanh cL_3}{c}$$

$$\$ = \frac{d}{X_1 + L_2 + X_3}$$

$$h_x = H\left(\frac{\sinh c(L_3 - x)}{\sinh cL_3}\right)\left(\frac{X_3}{X_1 + L_2 + X_3}\right)$$

Head beneath blanket at distance x from downstream toe . . . . . . h_x

Vertical permeability of blanket. . . . . k_b

The factor . . . . $C = \sqrt{\frac{k_b}{k_f z d}}$

Shape factor for use in seepage equation . . . . $\$$

Seepage per unit length of dam. $Q = \$ k_f H$

Figure B-9. Equations for semipervious blanket computations [39].

Appendix C

Discussion of SEEP/W Seepage Analyses and Example Problems

I. INTRODUCTION TO SEEP/W ANALYSES

Seepage analyses performed within the Geotechnical Engineering groups predominantly use the finite element numerical method computer program SEEP/W, which is part of the GeoStudio software package. The SEEP/W program can be used to model fluid flow and pore-water pressure distribution within porous materials such as soil and rock. In general, the SEEP/W program can be used for modeling a saturated-only flow or both a saturated and unsaturated flow condition. The typical saturated-only flow problem is a confined flow problem, such as seepage flow beneath a structure, as depicted in figure C-1. The saturated and unsaturated flow is an unconfined flow problem, such as a flow through an embankment dam, as depicted in figure C-2.

In addition to traditional steady state saturated flow analysis, the saturated/unsaturated formulation of SEEP/W makes it possible to analyze seepage as a function of time and to consider such processes as an embankment rapid drawdown and infiltration of precipitation. This transient feature allows analyzing such problems as the migration of a wetting front and the dissipation of excess pore-water pressure.

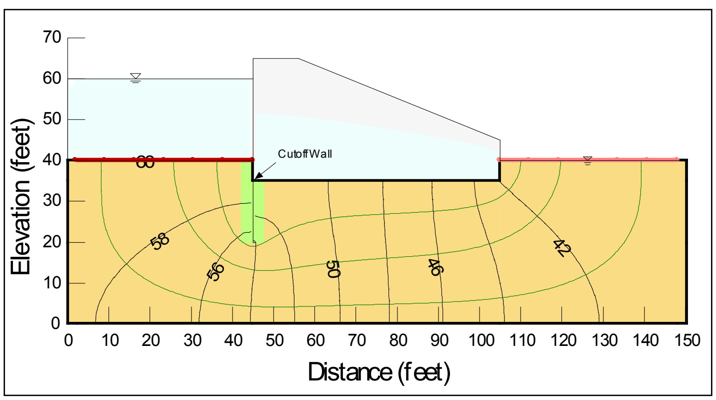


Figure C-1. Seepage flow beneath a concrete structure.

Flow is considered unconfined when the upper level of saturation (or phreatic surface) is not known. In unconfined flow problems, seepage analysis determines the location of the phreatic surface, which is the transition from positive to negative pore water pressures; therefore, it computes both saturated and unsaturated flow. The SEEP/W phreatic surface is not a flow boundary but is a line of zero pore-water pressure. Figure C-2 shows the saturated and unsaturated zones and the estimated location of a phreatic surface within an embankment.

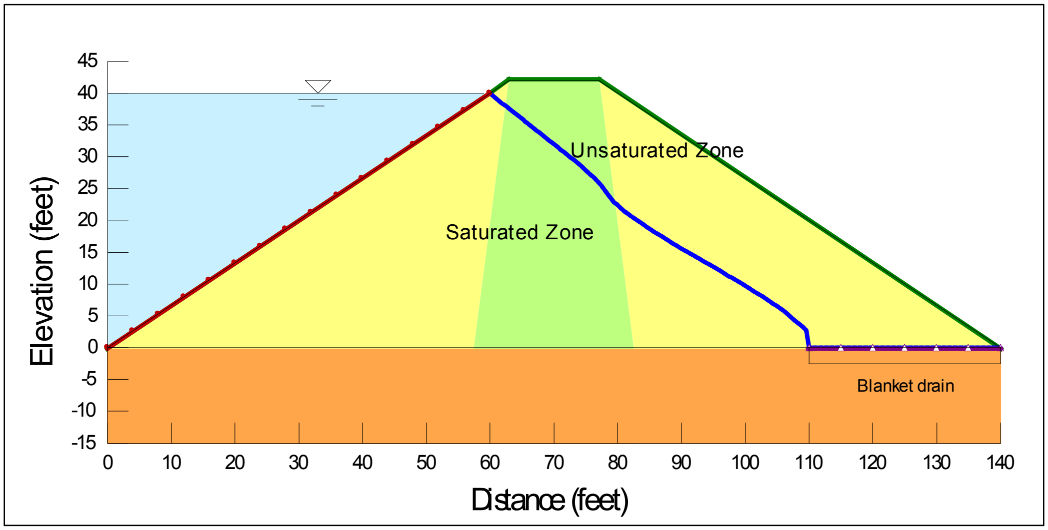


Figure C-2. Seepage flow through an embankment dam.

II. SEEP/W PROGRAM

A. General

The fundamental aspects of finite element modeling are discretization or meshing, and defining material properties and boundary conditions. Creating the finite element model includes selecting an appropriate geometry, dividing the model into appropriate regions, and creating the discretized mesh. The required input data include the specification of material properties to the various subregions of the domain, and the specification of the appropriate boundary conditions.

There are two types of seepage conditions that can be solved using the SEEP/W program: steady state and transient analyses. A steady state condition is independent of time; a situation where the state of the model is steady and not changing. A transient analysis is a condition that is always changing and is time-dependent. In order to move forward in time during a transient analysis, the user must provide initial conditions, as well as current or future boundary conditions. This can also be thought of in terms of loading. Steady state is a constant or steady loading, while transient is a loading that changes with time. Most seepage situations studied by Reclamation are transient because the reservoir level is usually changing over a season. A simplifying assumption is usually made that the loading is constant or steady, but this is a conservative assumption.

Defining a seepage model includes creating profile geometry, defining material properties, and defining boundary conditions. These tasks are summarized in subsequent sections.

B. Geometry

Profile geometry is defined as a series of geometric objects. These objects can be soil regions, circular openings, line objects, surface regions, and point objects.

Regions may be simple, straight-sided shapes like quadrilaterals or triangles or a free form as a multisided polygon; however, computational problems are reduced if the profile consists of simple triangles or quadrilaterals regions.

It is also possible for a point to exist within a region or outside of a region on its own. By default, a finite element "node" must exist at the location of all points, whether region corner points or free points. The advantage of using a "free point" is to ensure that a boundary condition can be applied at the desired location.

A free line is a line object that does not make up any part of a region edge. They can be very useful for applying anchors to a model or for specifying a geofabric or an insulation layer. They can also be used for creating structural components that are partially in the soil and partially outside the soil.

A circular opening is a type of region that "floats" over the top of another soil region. This region can be dragged to a different location, or its circumference point can be moved to change the size of the opening.

C. Finite Element

One of the main features of a finite element model is the nodes. All finite element equations are formed at the nodes. All elements common to a single node contribute to the characteristics and coefficients that exist in the equation at that node; therefore, the seepage equation is developed for each node, and the material properties which are used within the equations are contributed from the surrounding elements.

The SEEP/W user's guide indicates that nodes are used for the following purposes:

1. The positions of the nodes in a coordinate system are used to compute the geometric characteristics of the element – such as length, area, or volume.

2. The nodes are used to describe the distribution of the primary unknowns within the element, and the primary field variable is the hydraulic head or pore-water pressure.

3. The nodes are used to connect or join all the elements within a domain. All elements with a common node are connected at that node. It is the common nodes between elements that ensure compatibility.

In the current version of the GeoStudio 2007 software, all meshing is fully automatic. There is no capability to draw individual "finite elements." However, there are no longer concerns that the mesh will be incompatible across different regions or whether the material properties or boundary conditions will disappear if the mesh is changed. With the automatic mesh generation algorithms, the user can define a single global element size parameter. The user may alter the size of the elements at a global level for the entire mesh, within any one or more regions, or along a line or around a point.

D. Soil Properties

The most important soil property used in seepage analysis is the hydraulic conductivity or coefficient of permeability (including anisotropy). In soils, the hydraulic conductivity and the water content (or water stored) change as a function of pore-water pressure. The ability of a soil to transport or conduct water under both saturated and unsaturated conditions is reflected by the hydraulic conductivity function. Soil, which consists of a collection of solid particles and interstitial voids, has pore spaces or voids that can be filled either with water or air, or with a combination of both. Degree of saturation of a soil is equal to the volumetric water content (VWC) over the porosity of soil. In a saturated soil, all the voids are filled with water, and the volumetric water content is equal to the porosity of the soil. In unsaturated soil, the volume of water stored within the void will vary depending on matric suction within the pore water. Since there is no fixed water content in time and space, a function is required to describe how water content changes with different pressures in the soil.

The VWC function describes the capability of the soil to store water under changes in matric pressure. Figure C-3 shows three main features that characterize the volumetric water content function.

The three key features are the air-entry value (AEV), the slope of the function for both the positive and negative pore-water pressure ranges (designated as m_w), and the residual water content or saturation, (or S_r). The AEV corresponds to the value of negative pore-water pressure when the largest voids or pores begin to drain freely. It is a function of the maximum pore size in a soil and is also influenced by the pore-size distribution within a soil. Soils with large, uniformly shaped pores have relatively low AEV values.

Each type of soil has a different volumetric water content function; sand will drain faster than silt, and silt needs less time to drain than a clay soil. Figure C-4 shows typical values of the volumetric water content functions for sand, silt and clay soils.

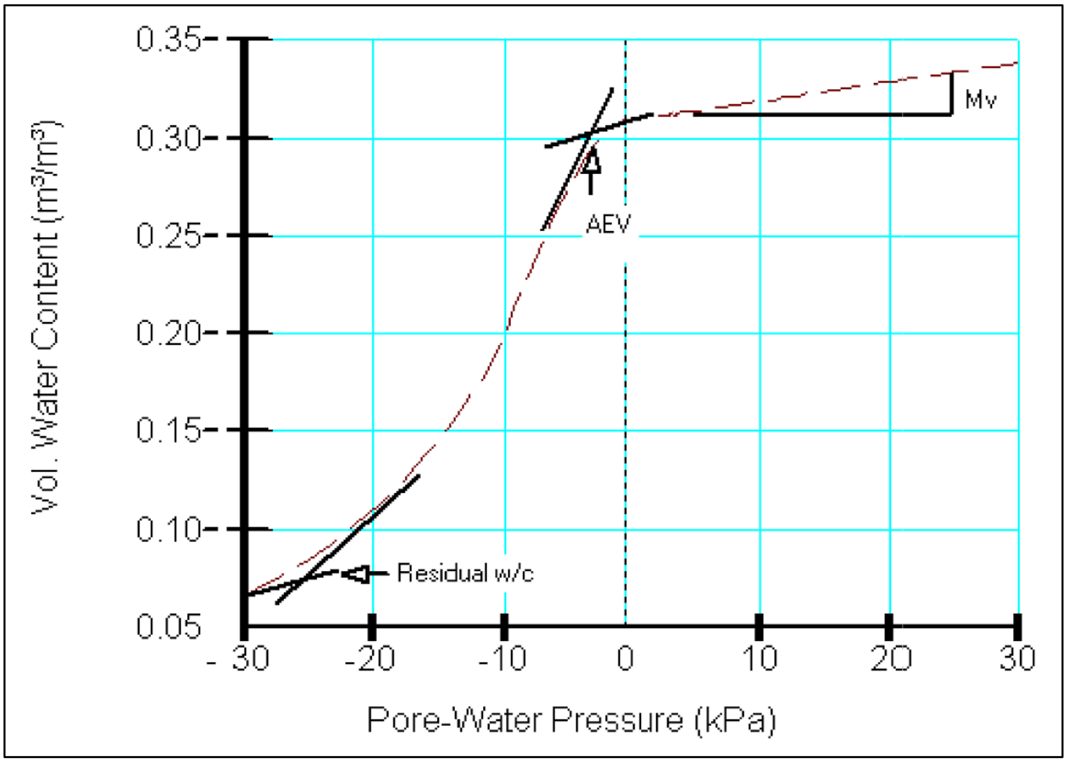


Figure C-3. Volumetric water content (storage) function.

For steady state seepage analysis, seepage is independent of time; therefore, the VWC function is not required. For transient analysis, the volumetric water content function is one of the required input parameters.

Since obtaining the VWC can be time consuming, SEEP/W users can estimate a VWC function using any of four methods summarized below:

1. Closed Form Option 1 (Fredlund and Xing, 1994)
   This method is a closed-form solution, which was based on a group of three curve fitting parameters of a, n, m, that can be used to develop the volumetric water content function for all possible negative pressures between zero and minus one million kilopascals.

2. Closed Form Option 2 (Van Genuchten, 1980)
   Van Genuchten proposed a four-parameter equation as a closed form solution for predicting the volumetric water content function.

   Detailed information about the two options above are documented in the "Seepage Modeling with SEEP/W - an Engineering Methodology," GeoStudio 2007, Fourth Edition, May 2009.

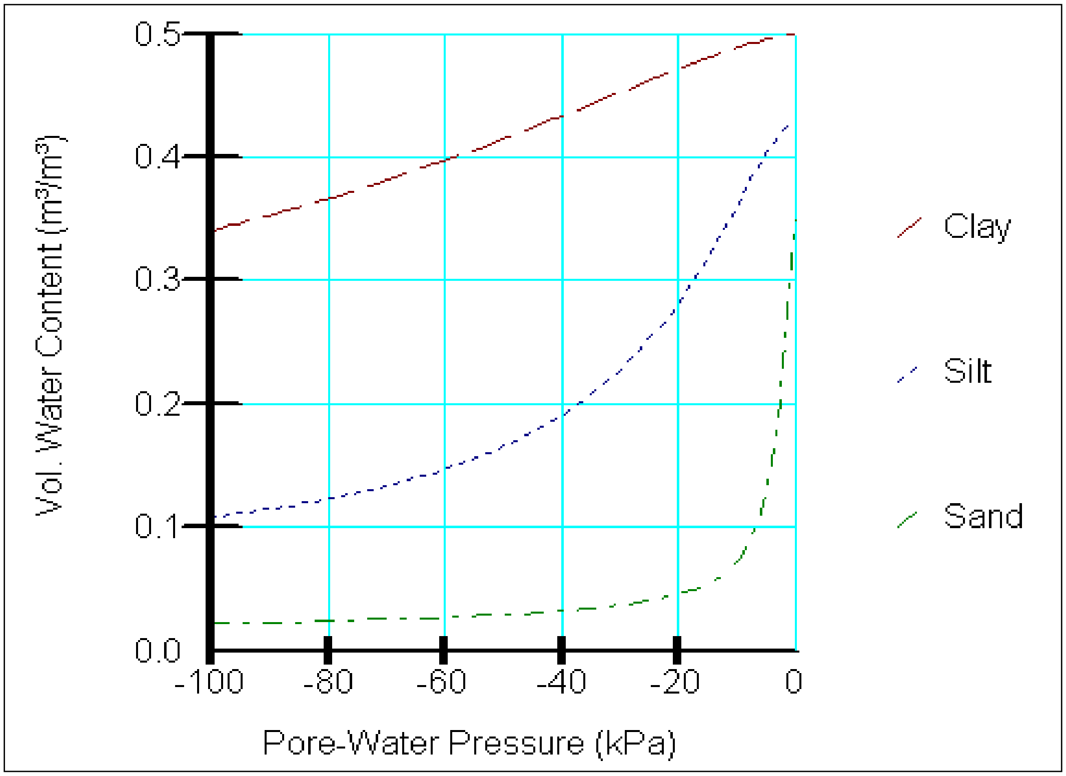


Figure C-4. Typical values of volumetric water content for sand, silt, and clay soils.

3. Estimation Method 1 (grain size - Modified Kovacs)
   This method estimates a data point function using a predictive method based on grain size. This method predicts the volumetric water content function using basic material properties which can be useful, particularly for preliminary analysis.

4. Estimation Method 2 (sample functions)
   SEEP/W provides several "typical" water content functions for different types of soils. In using these sample functions, it is up to the user to specify the saturated water content and the residual water content (if any) based on the user understanding of field conditions. These functions are provided as a means of letting the user set up some test models quickly, change functions easily, decide how sensitive the results are to function shape, and, ultimately, to decide if there is a need to spend more time and money obtaining more accurate data.

Figure C-5 shows "typical" volumetric water content functions for different types of soils (over a greater range of matric suction pressures than shown in figure C-4).

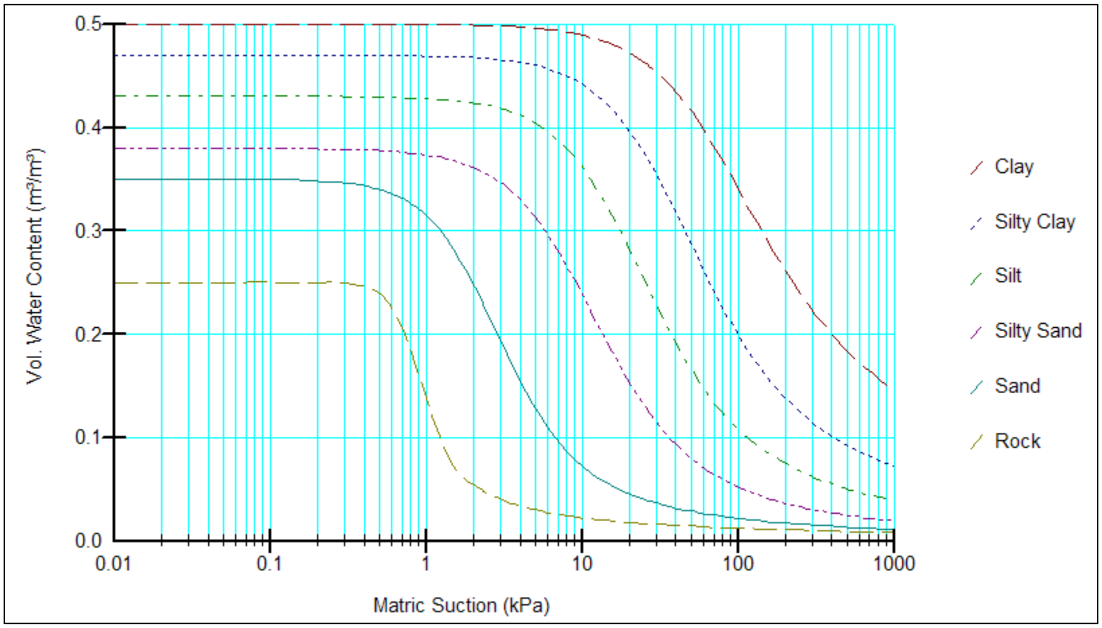


Figure C-5. Typical volumetric water content functions of soils.

Similar to obtaining the VWC, measuring the hydraulic conductivity function is also a time-consuming, potentially expensive procedure. There are functions that can be readily developed using one of several predictive methods that use either a grain-size distribution curve or a measured volumetric water content function and the saturated hydraulic conductivity.

SEEP/W has built-in predictive methods that can be used to estimate the hydraulic conductivity function once the volumetric water content function and a K_{sat} value have been specified. Details on how to estimate the hydraulic conductivity can be found in the SEEP/W Engineering Methodology manual.

Figure C-6 shows examples of hydraulic conductivity functions for different soil materials.

In the SEEP/W program, there are four different models to choose from when defining material properties for analysis.

1. "None" is for specifying regions that are not going to be included in the analysis.

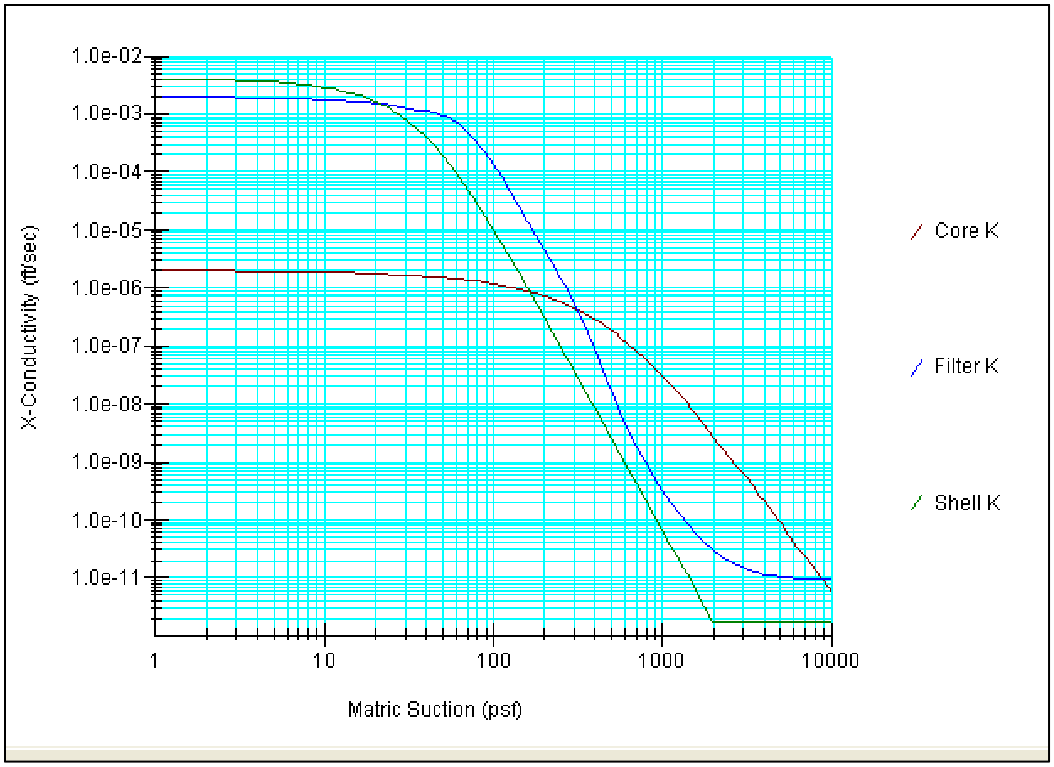


Figure C-6. Example hydraulic conductivity of soils.

2. The "saturated/unsaturated model," in which the values of hydraulic conductivity or permeability value, ratio, and direction need to be specified. In SEEP/W, permeability in the horizontal direction (x-coordinate) is the input value. The permeability ratio is the ratio of the vertical permeability to the horizontal permeability. Another direction of flow, other than in the x-y coordinate direction, can also be specified by user.

3. The "saturated only model," which only needs a constant value of saturated permeability (K_{sat}); therefore, no permeability function is needed. The "saturated only" soil model should only be used to specify soil regions that will always remain below the phreatic surface, from the left end (upstream) to right end (downstream) of a seepage model.

4. The "interface model," which allows the user to assign a material model to a line and to give that line a thickness. It can be used to simulate geomembranes, wick drains, or cutoff walls. The interface model would have permeability values that are both normal and tangent to the direction of the line.

E. Boundary Conditions

Seepage analysis solutions are a direct response to the boundary conditions. A boundary condition could be the total hydraulic head difference between two points or some specified rate of flow into or out of the system, but not both.

In SEEP/W analyses, boundary conditions can only be specified by either of two options: hydraulic head (H) or flow quantity (Q). Only one of these boundary conditions can be specified (either the H or the Q) at a boundary. Head boundaries can be applied as total head, pressure head, or time/function of head. Flux (flow) boundaries can be applied as fixed values of total or unit flux, or a time-dependent function of total or unit flux.

An important concept to recognize is that when an H is specified at the node, the solution will provide the computed flux, Q. Alternatively, when the Q is specified, the solution will provide the computed total head, H.

Other types of boundary conditions include: (1) a source or sink, and (2) "far field" boundaries. A source boundary condition could be an injection well. A typical sink might represent a drain at some point inside a mesh. The important concept about sources and sinks is that they represent flow into or out of the system.

In many cases, the boundary conditions far away from the main point of interest are required, such as the boundary conditions far away from a pumping well or from an excavation. These are referred to as “far field” boundary conditions. This boundary condition is located at the edge of the profile, which is assumed to be extending far away from the main problem. A far field boundary condition is called an infinite element; the node of the element at the profile end is assumed to be at infinity.

For transient analyses, boundary conditions are specified as a function of time (head versus time) or in response to flow amounts exiting or entering the flow region (head versus volume).

III. ANALYSIS TYPES

There are two fundamental types of finite element seepage analyses: steady state and transient. Steady state analysis assumes that the water pressure and water flow rate have reached a steady condition and are independent of time, even though, in reality, a steady state will never be reached. In a steady state analysis, there are two choices of boundary conditions: a constant pressure (or head) and a constant flux rate. The flux rate can be specified as a total nodal flux or a unit flux applied to an element edge.

Transient analysis is a time-dependent analysis, which allows the user to apply a fixed or time-dependent boundary condition and/or to compute the change in pore-water pressure at different times. Time step information has to be entered for this analysis. Because of time dependent analysis, an initial condition must be defined before the transient analysis can be performed.

For a transient analysis, it is essential to define the initial (starting) total head at all nodes. SEEP/W allows the user to specify the initial conditions by either reading the data from an initial conditions file created in a separate analysis or by drawing the initial water table position. It is important to recognize that the initial conditions for a transient analysis can have a significant effect on the solution. Unrealistic initial conditions will lead to unrealistic solutions that may be difficult to interpret, especially in the early stage of the transient analysis. In most cases, the initial conditions are established by running a steady-state analysis.

IV. ANALYSIS RESULTS

Seepage analysis results are summarized in the "Contour" section of the SEEP/W program. The main objectives of a seepage analysis are frequently to obtain a plot of pore-water pressure distribution and to determine the quantity of seepage.

In SEEP/W finite element analyses, all output data for nodes and gauss points (of an element) anywhere in the model are accessible using the "View Results Information" command. Figure C-7 shows typical SEEP/W output results for a node located on a piezometric line (zero pore pressure).

With SEEP/W, flow quantities are computed by defining a water flux. The location of the water table, as computed by SEEP/W, is drawn in contour along an isoline (phreatic line), where the water pressure is zero. Figure C-7 shows that there is a seepage flow out (negative water flux) at that point.

Figure C-8 shows the output data from one gauss point of an element below the phreatic line.

SEEP/W does not create a true flow net; however, it does compute and display elements of flow net principles. Instead of flow lines, SEEP/W creates flow paths, which are lines that an imaginary droplet of water would follow from entrance to exit; they are not flow lines in the true context of a flow net. A flow path is not a flow channel but, rather, a seepage path.

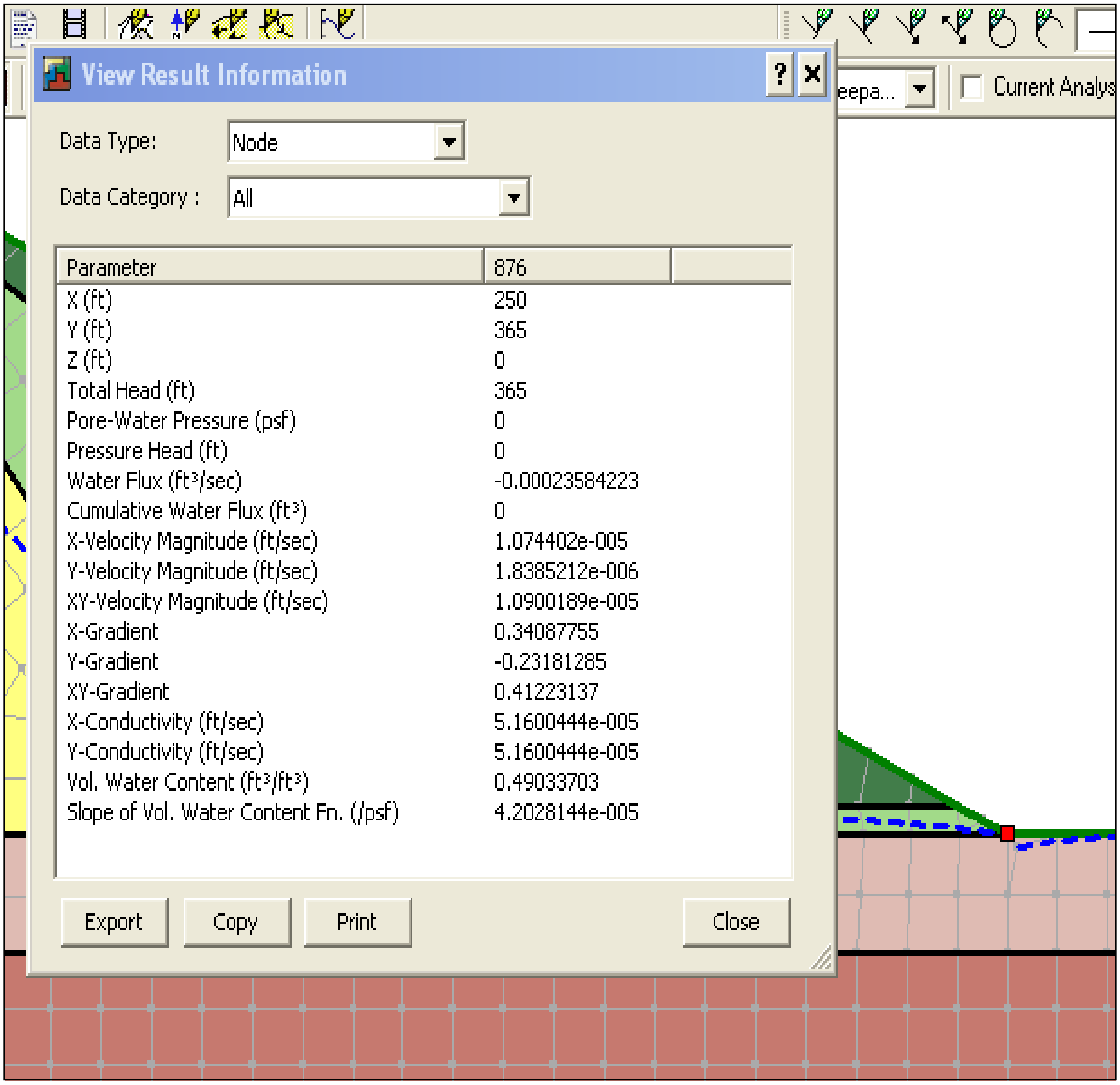


Figure C-7. Seepage analysis output results – node with zero pressures.

In a saturated / unsaturated flow system, water can flow from the saturated to the unsaturated zone, and vice versa. Flow can take place across the phreatic surface, which is the line of zero water pressure. Consequently, a flow path may cross the phreatic surface as illustrated on figure C-9.

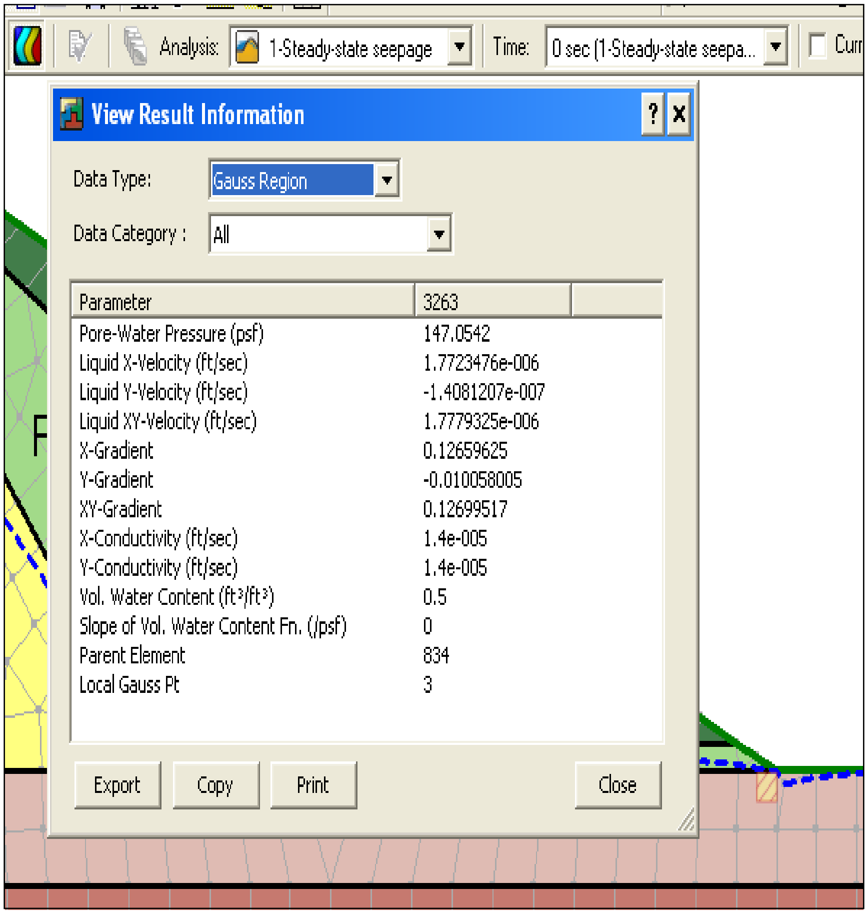


Figure C-8. Seepage analysis output results – at the gauss region.

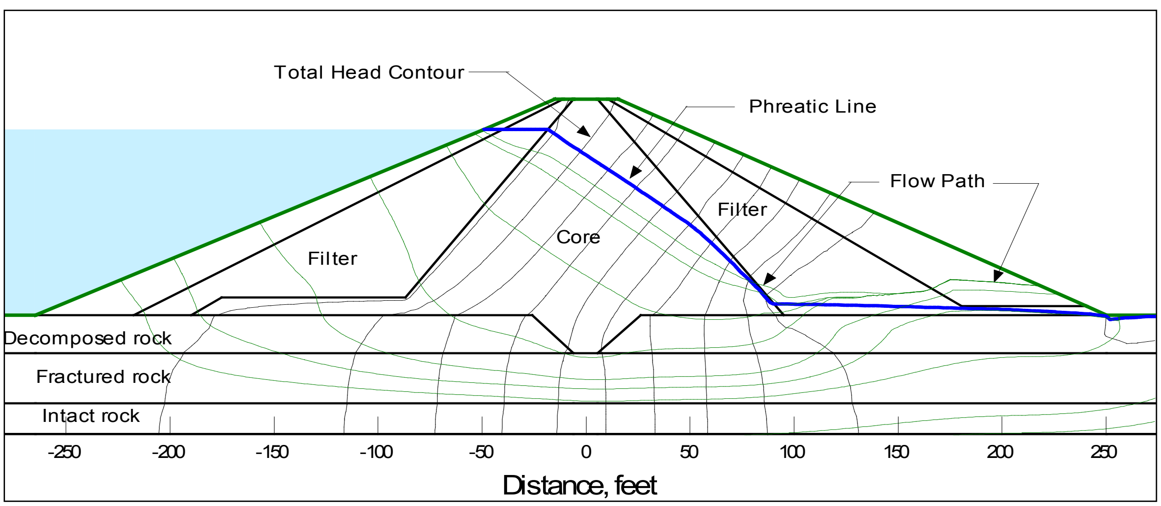


Figure C-9. Phreatic line, total head, and flow paths within the dam.

V. EXAMPLES

Example problems are divided into two categories of seepage flows: confined and unconfined flows. An example of confined flow is seepage flow within the (saturated) foundation beneath a concrete structure. Unconfined flows consist of steady state and transient analyses.

The list of examples for both confined and unconfined flows includes the following:

1. Seepage flow under a structure, with or without cutoff wall
2. Existing dam, steady state analysis – calibration run, rapid drawdown
3. New dam, transient analysis – first filling, steady state, and drawdown
4. Embankment core with varying permeability
5. Drain – defined using different types of boundary conditions
6. Design of relief well spacing

Example 1 - Seepage Flow Under a Structure

I. Introduction

This example represents a confined flow analysis, where seepage is flowing through the pervious foundation of a concrete dam. The foundation upstream of the structure is below the reservoir, and tailwater is at the level of the foundation surface downstream of the dam structure. Therefore, the foundation material is in a saturated state, having a saturated permeability.

In this example, two steady state seepage analyses were performed; the first analysis was for seepage under concrete dam without any cutoff, and second analysis evaluated the effects of a cutoff wall at various points beneath the dam. The cutoff wall is defined as an interface material. Results of both analyses including total head, exit gradient, and uplift pressure are summarized.

II. Analysis Data

Figure C-1.1 presents the SEEP/W model of the concrete dam with an upstream cutoff wall. The upstream and downstream edges of the model were specified as infinite regions that would be far enough from the main area of interest to avoid significantly influencing the results.

The boundary condition on the upstream side of the model is a total head value equal to the elevation of the water in the reservoir. At the downstream side, the boundary condition is set to a pressure head equal to zero – which indicates full saturation downstream with no tailwater elevation above the ground surface. The cutoff is installed to a depth of 15 feet beneath the dam and is modeled using interface elements along a line. This interface was assumed to have a permeability value close to zero; therefore, it must be a no-flow feature, and this is achieved by setting the tangential and normal conductivity along the interface elements to values of zero.

The hydraulic conductivity of the homogeneous foundation material is assigned as $1x10^{-3}$ feet per second. Because the soil always remains saturated, the “saturated only” permeability was assigned to the foundation material. This analysis is a confined flow analysis, so no permeability function is required.

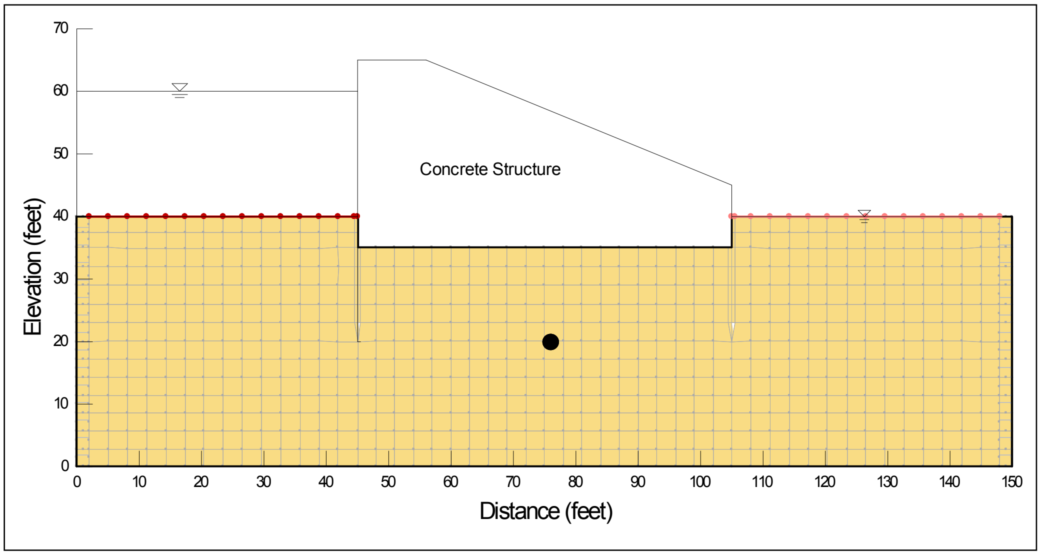


Figure C-1.1. SEEP/W model including boundary conditions.

III. Analysis Results

A. Dam Without Cutoff Wall

Assuming there is no cutoff wall, the interface region is modeled as a soil region. Figure C-1.2 shows estimated total heads within the foundation soil from the analysis.

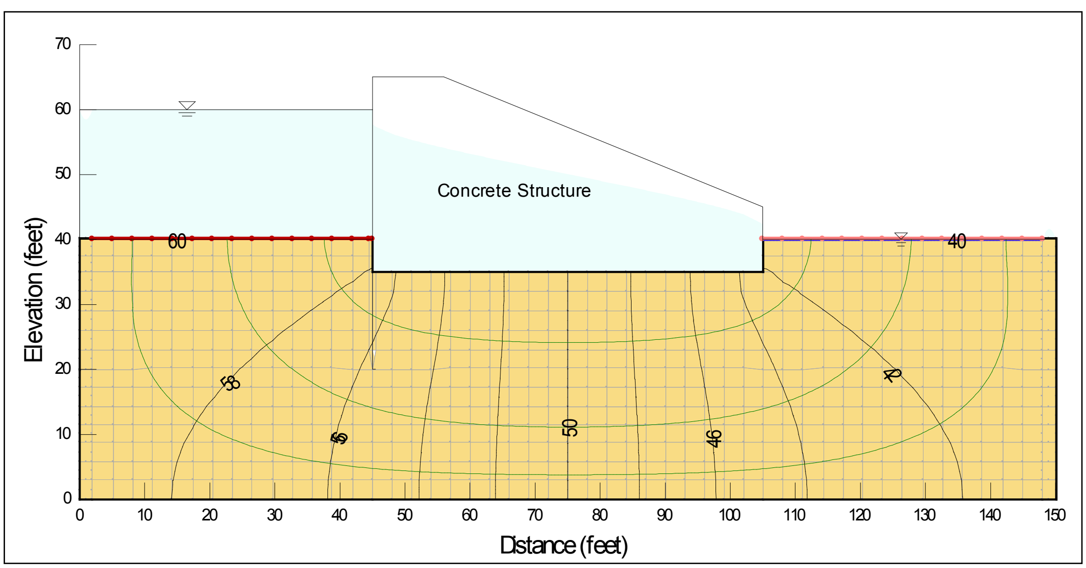


Figure C-1.2. Total head contours – dam without cutoff wall.

B. Dam With Cutoff Walls

Three different cutoff wall configurations were modeled: a cutoff at the upstream end of the structure, a cutoff at the downstream end of the structure, and cutoffs at both the upstream and downstream ends of the structure. Figures C-1.3 through C-1.5 show the resulting total head values, respectively.

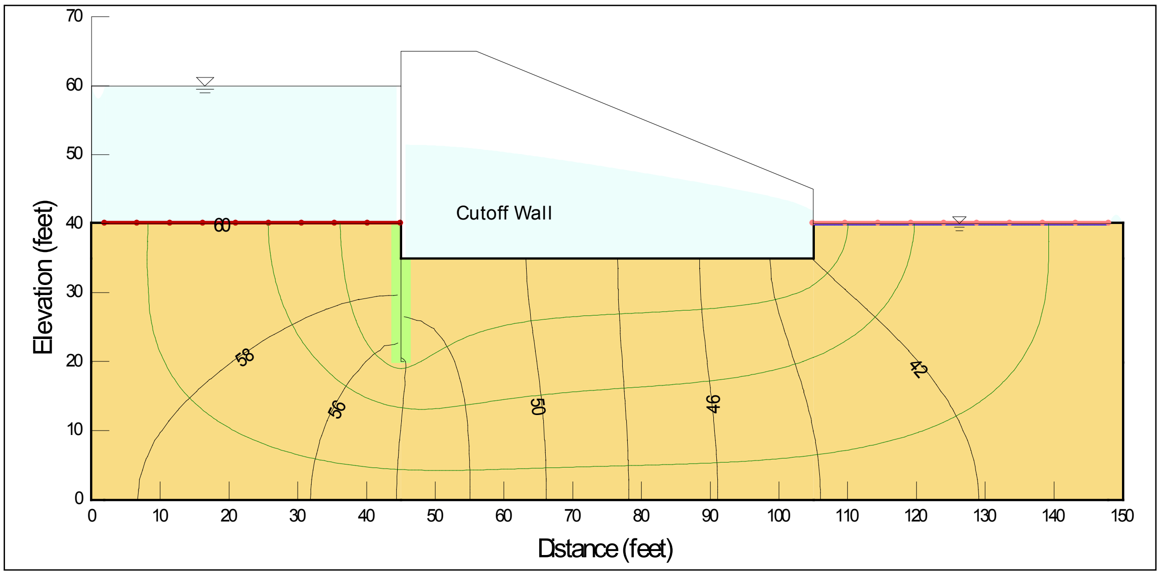


Figure C-1.3. Total head contours – dam with upstream cutoff wall.

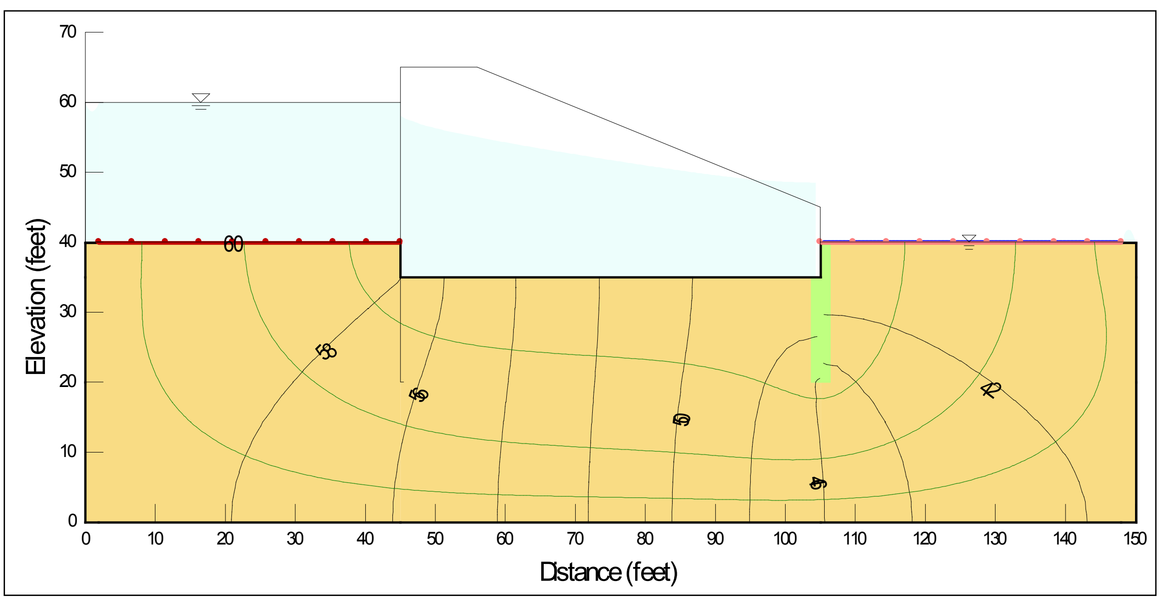


Figure C-1.4. Total head contours – dam with downstream cutoff wall.

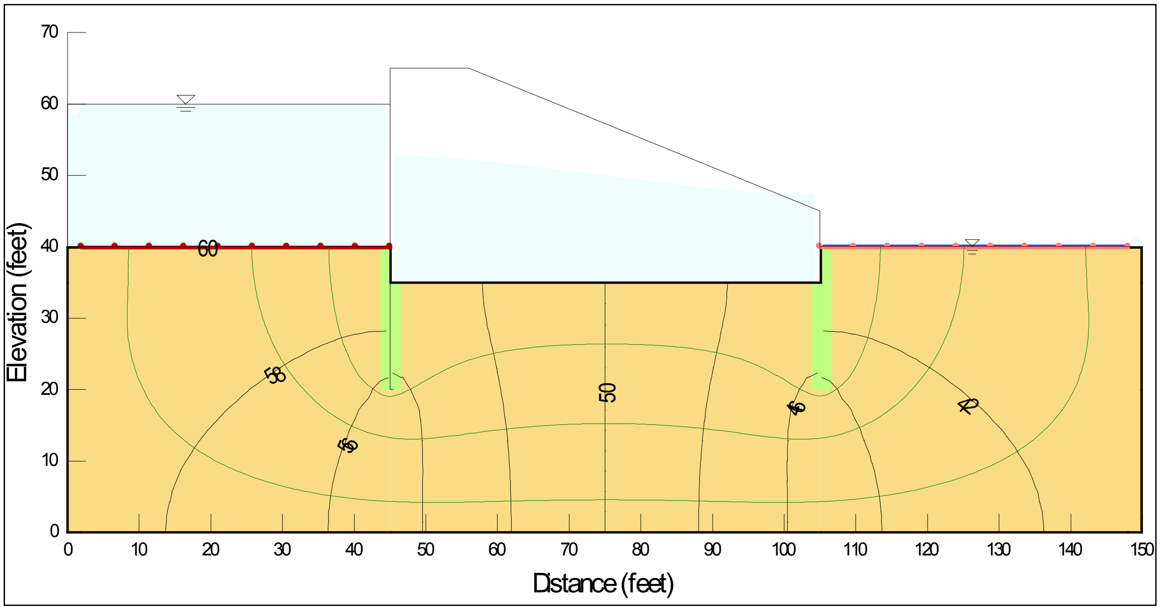


Figure C-1.5. Total head contours – dam with upstream and downstream cutoff walls.

Although the total head contours do not suggest dramatic differences at the downstream toe of the structure, contours of gradients at nodes surrounding the downstream toe show the effects of a cutoff much more clearly. Figure C-1.6 shows the computed vertical (y) gradients at the downstream toe without any cutoff wall, while figure C-1.7 shows the computed vertical gradients with a downstream cutoff wall.

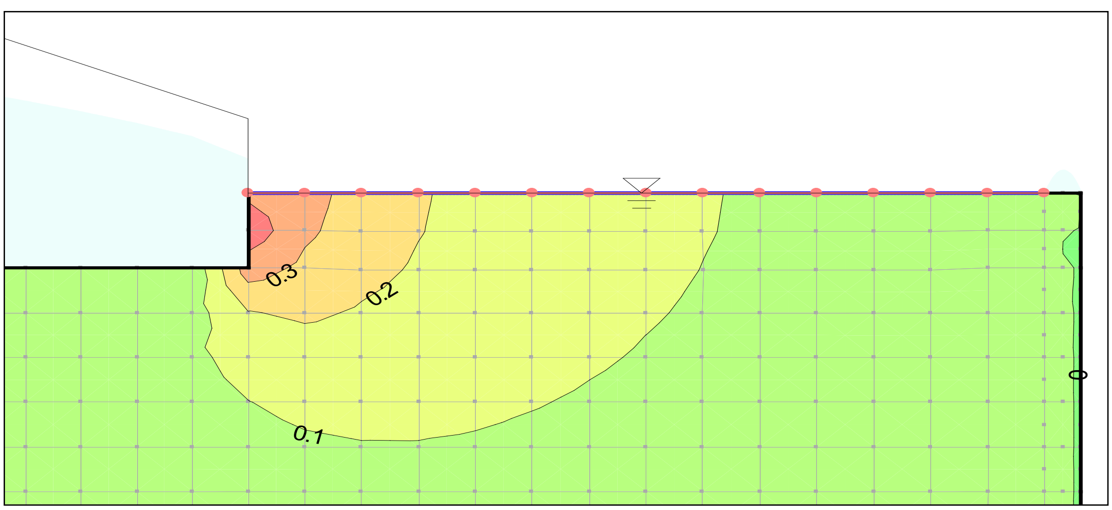


Figure C-1.6. Seepage gradient contours – dam with no cutoff walls.

For the structure with downstream cutoff wall, the y-gradients at the downstream structure (shown in figure C-1.7) were less than the y-gradients shown in figure C-1.6.

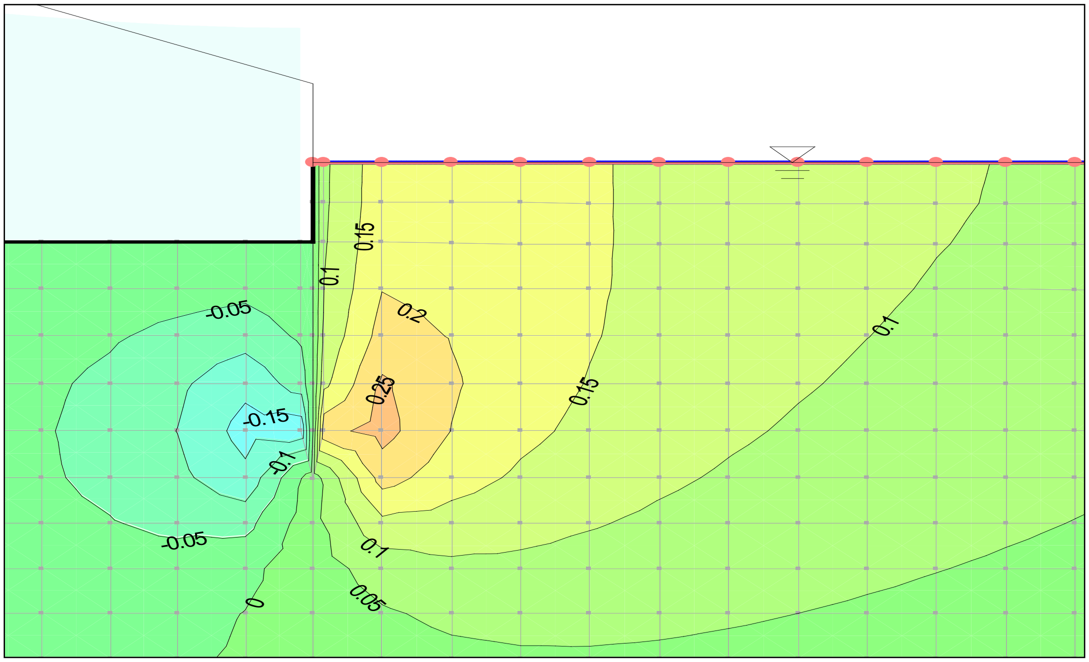


Figure C-1.7. Seepage gradient contours – dam with downstream cutoff wall.

The results show that the dam with a cutoff structure placed on the downstream edge of the dam would have smaller vertical exit gradients compared to the dam without a cutoff wall. Table C-1.1 provides a summary of the gradients calculated from the four different alternatives included above.

Table C-1.1. Calculated X-, Y-, XY- Gradients at Downstream Toe – Different Cutoff Location

Description	No Cutoff	Upstream Cutoff	Downstream Cutoff	Upstream and Downstream Cutoff
X-gradient	0.195	0.163	0.003	0.003
Y-gradient	0.498	0.417	0.185	0.160
XY-gradient	0.535	0.448	0.185	0.160

Example 2 – Existing Embankment, Steady State, and Rapid Drawdown

I. Introduction

This example presents an analysis of an existing embankment dam. The analysis is an unconfined flow analysis, which consists of steady state and transient analysis. The dam is a zoned earthfill structure with a maximum structural height of 185 feet. The width of the dam crest at the typical section is 30 feet. The dam embankment consists of three zones. Zone A is the outer shell, placed at the downstream and upstream surfaces, comprised of "dirty rockfill" from excavation in the river channel and material from preexisting stockpiles. Zone B is a transition (filter) zone, upstream and downstream of the core zone, consisting of unprocessed sand, gravel, and cobbles from excavation in the river channel. The core, Zone C, consists of decomposed granite and fine-grained material from river channel excavation. The upstream and downstream slopes of the dam are 2.5 (H) to 1 (V) and 2 (H) to 1 (V), respectively. The crest elevation is 483.

The embankment is founded on a granite formation. The upper layer is mostly decomposed to intensely weathered granite with a thickness of a few feet to about 45 feet. The middle layer is moderately weathered granite. The bottom layer of granite is a sound bedrock or unweathered granite. Figure C-2.1 shows the SEEP/W model of the typical section of the dam.

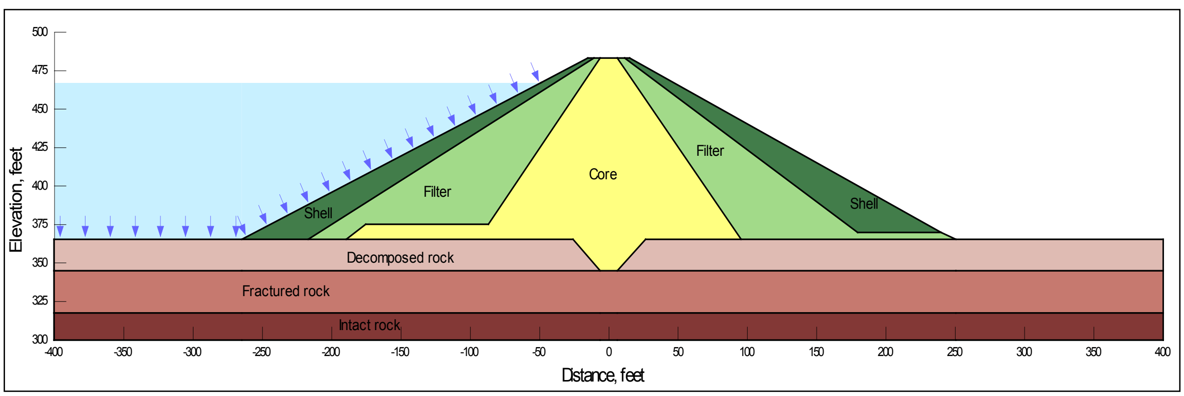


Figure C-2.1. Typical section of example embankment dam.

II. Existing Data

During a past field exploration, seven drill holes were drilled from the dam crest. Mostly only the Zone C core materials were encountered within these seven drill holes. Laboratory tests indicate that more than 80 percent of the soil samples are nonplastic silty sand (SM); some areas have clayey fines and are classified as SC-SM.

Piezometers were placed in the drill holes and within the embankment core zone. For reservoir at elevation 467, the highest piezometer level was about elevation 422, and the maximum seepage at the downstream toe was measured at about 4.0E-04 cubic feet per second per foot ($ft^3/s/ft$).

There are permeability data that were obtained from borrow area and foundation materials during design. Saturated permeability of borrow materials ranged between 0.016 and 0.35 feet per day (ft/d). Decomposed granite had a permeability of 0.04 up to 61 ft/d.

To simplify this example, the upper foundation at the downstream area was assumed to have saturated permeability only, since the toe drain was only a few feet below ground surface.

Permeability functions of embankment zones were estimated using the volumetric water content functions, as shown in figure C-2.2.

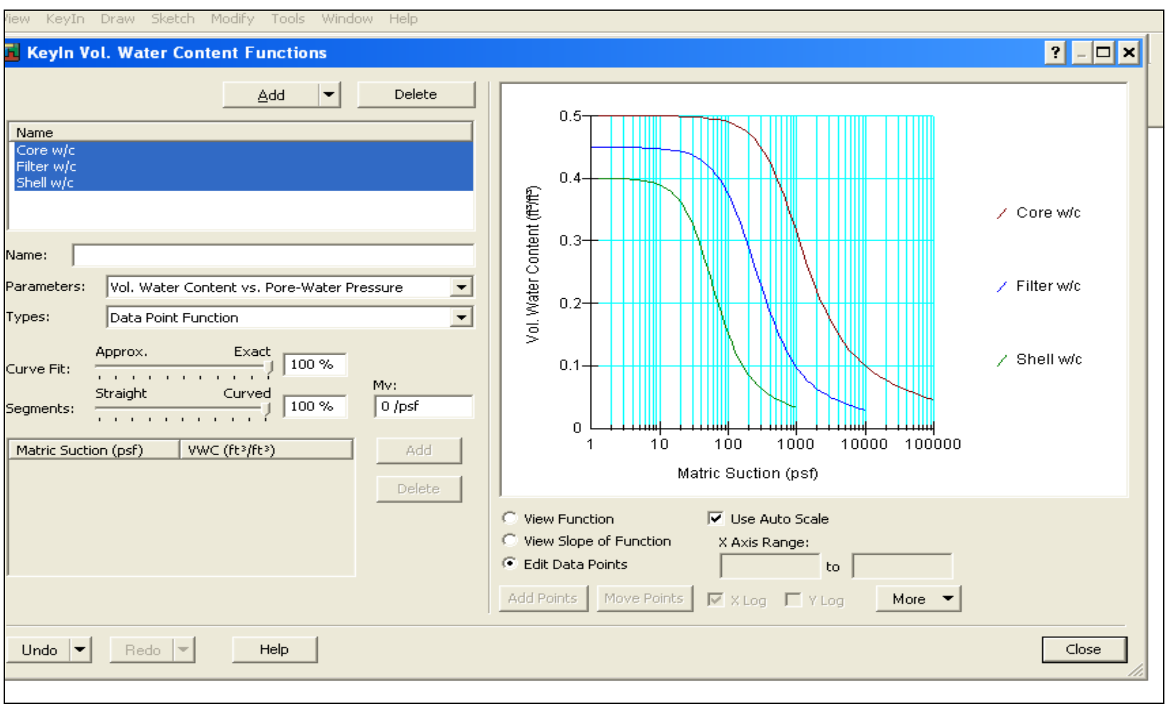


Figure C-2.2. Volumetric water content – embankment materials.

III. Analysis

Several calibration SEEP/W runs were performed using the existing permeability values of the embankment and foundation materials, which resulted in a phreatic surface and quantity of seepage close to the available data. Results of the final run, shown in figure C-2.3, estimated total head at the piezometer location at about elevation 422. Seepage quantity was estimated to be about 3.9E-04 ft^3/sec/ft.

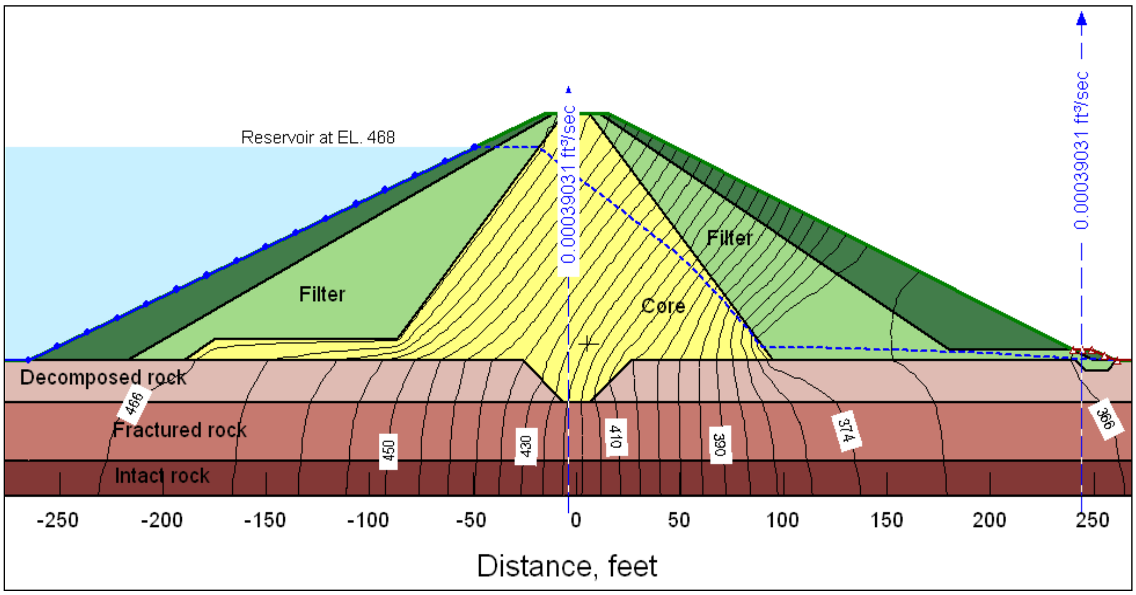


Figure C-2.3. Output of calibration run.

The estimated permeability values from the calibration runs were then used to estimate the seepage for a normal reservoir level at elevation 468. Figure C-2.4 shows that the total head and the seepage quantity are a little higher than from the calibration run.

Figure C-2.5 shows the estimated permeability functions of the embankment materials from the calibration run.

Assuming that the reservoir had to be lowered from elevations 468 to 418 within 10 days, an analysis was performed for a 5-feet-per-day rapid drawdown. Figure C-2.6 shows the phreatic lines from elevation 468 down to elevation 418.

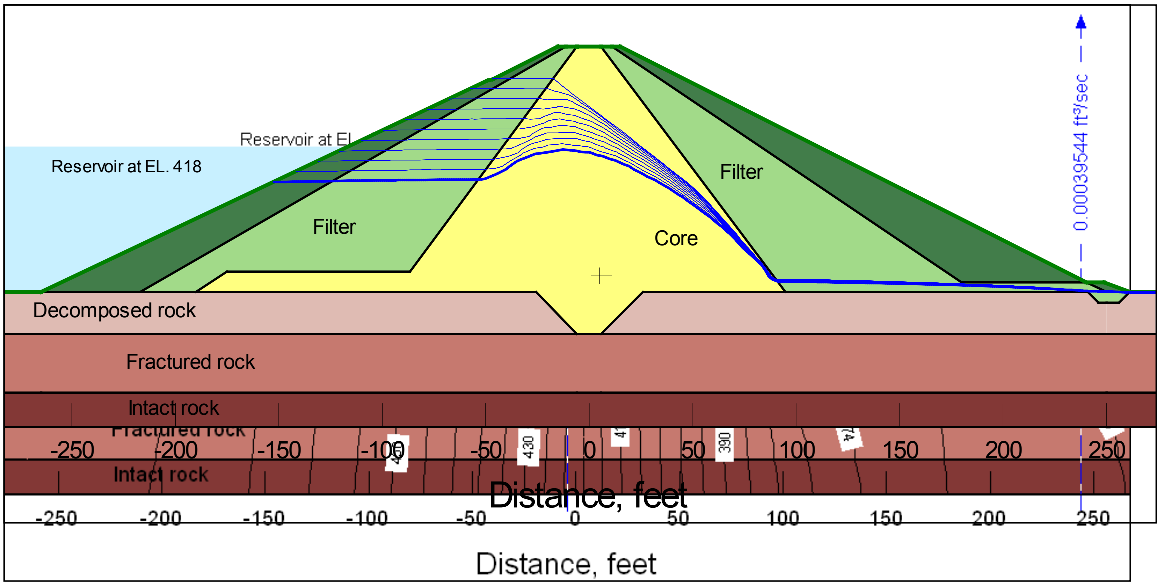


Figure C-2.4. Output run for reservoir at elevation 468.

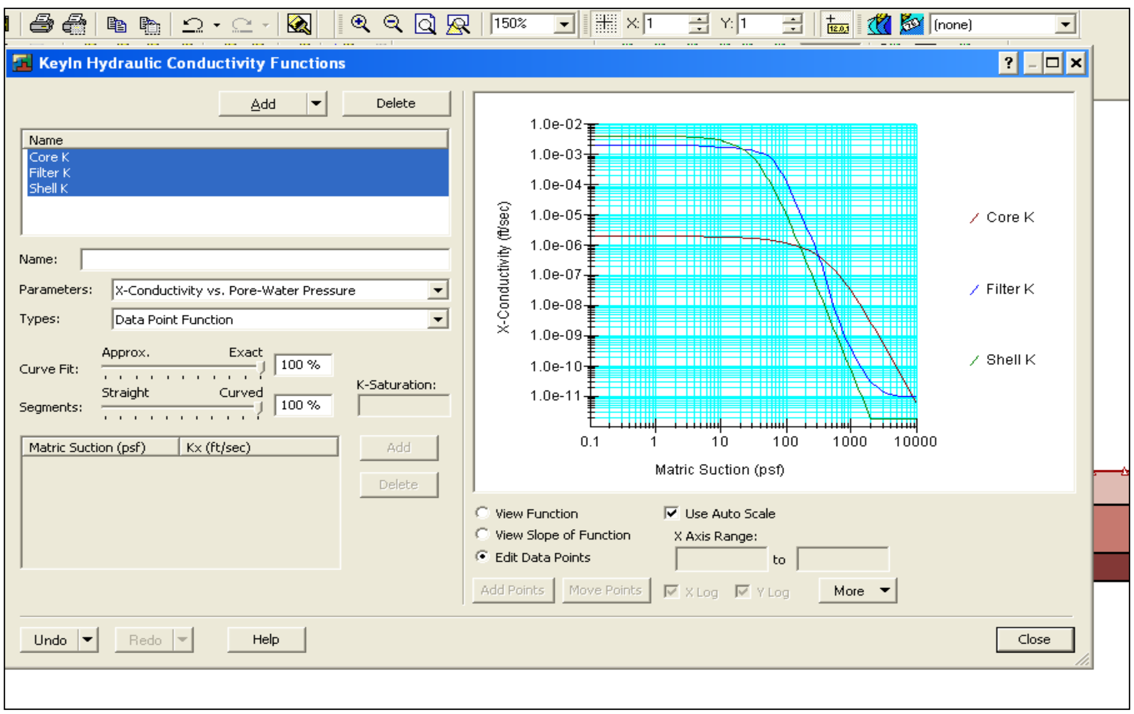


Figure C-2.5. Permeability function used for calibration run.

Figure C-2.6. Phreatic lines from reservoir elevation 468 to elevation 418.

Figure C-2.7 shows total head contours and vectors of seepage flows from the rapid drawdown case. Based on the location of total head, seepage through the foundation is flowing downstream, and most embankment seepage is flowing upstream.

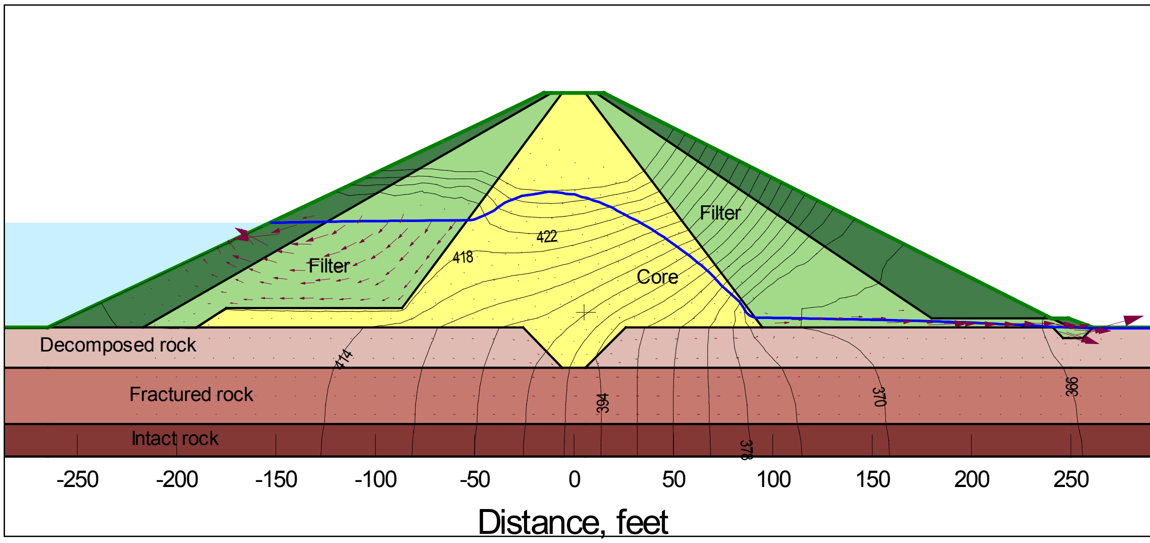


Figure C-2.7. Phreatic lines from reservoir elevation 468 to elevation 418.

Example 3 – New Dam – First Filling and Steady State

I. Introduction

The objective of this example is to illustrate the modeling of a new embankment at a first filling stage, period of a full reservoir, and drawdown of a reservoir. The boundary conditions are transient for all of these stages.

The embankment model shown in Example 2 is assumed to be a new dam structure, and the same geometry and permeability values will be used for this analysis.

II. Analysis Data

The new reservoir is assumed to be filled at a rate of one ft/d; therefore, units for this analysis would be in ft/d.

Figure C-3.1 shows the SEEP/W model used for analysis.

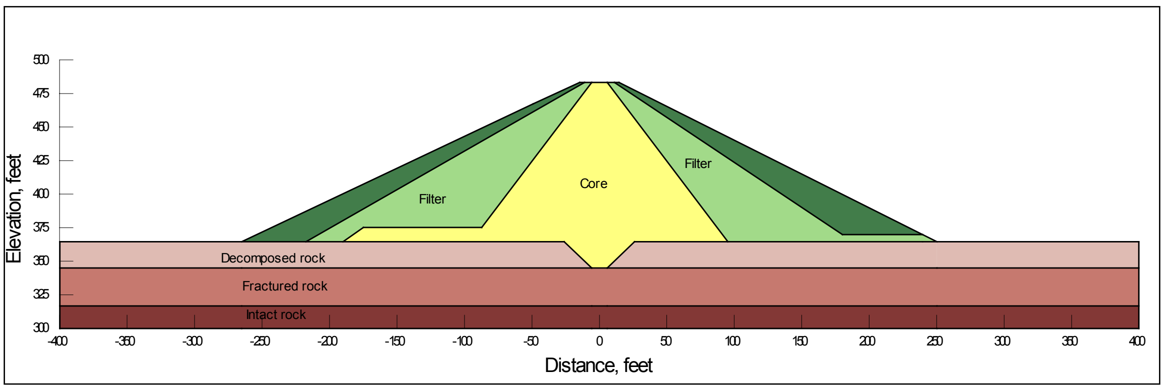


Figure C-3.1. SEEP/W profile of an empty reservoir at new dam.

The three foundation layers are assumed in saturated stage, so they were assumed to have saturated permeability only. Thus, no permeability functions are estimated for these foundation layers.

The embankment zones, core, filter, and shell would have permeability functions, which were estimated from VWC of typical soil materials. Figure C-3.2 shows the VWC function for the embankment materials, and figure C-3.3 shows the permeability functions.

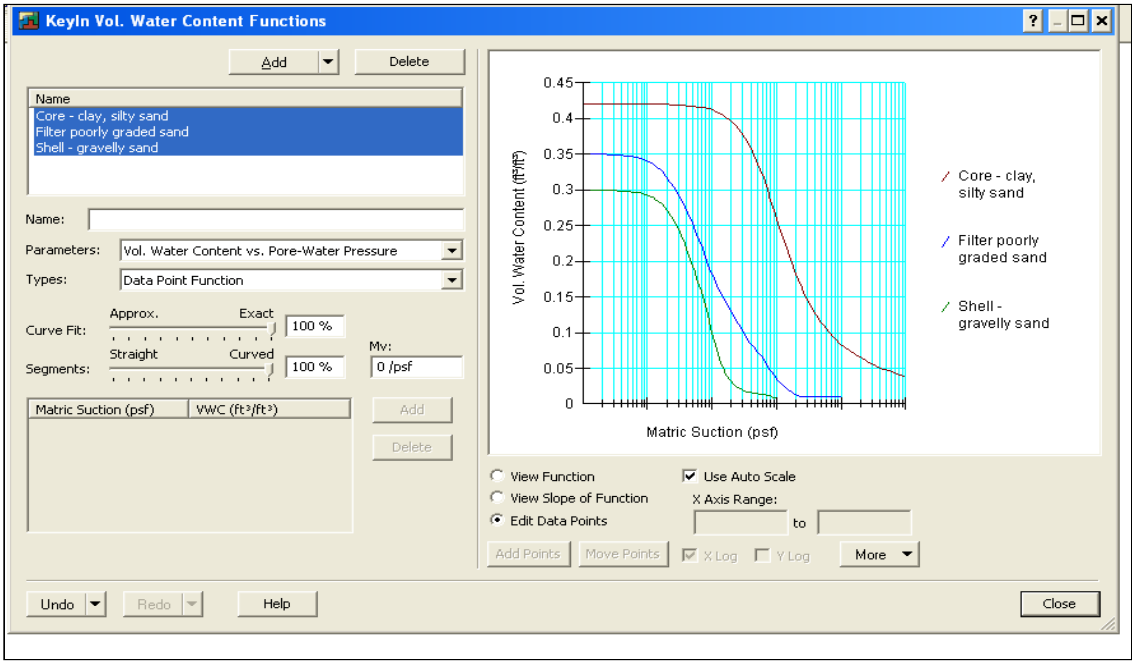


Figure C-3.2. VWC of three embankment materials.

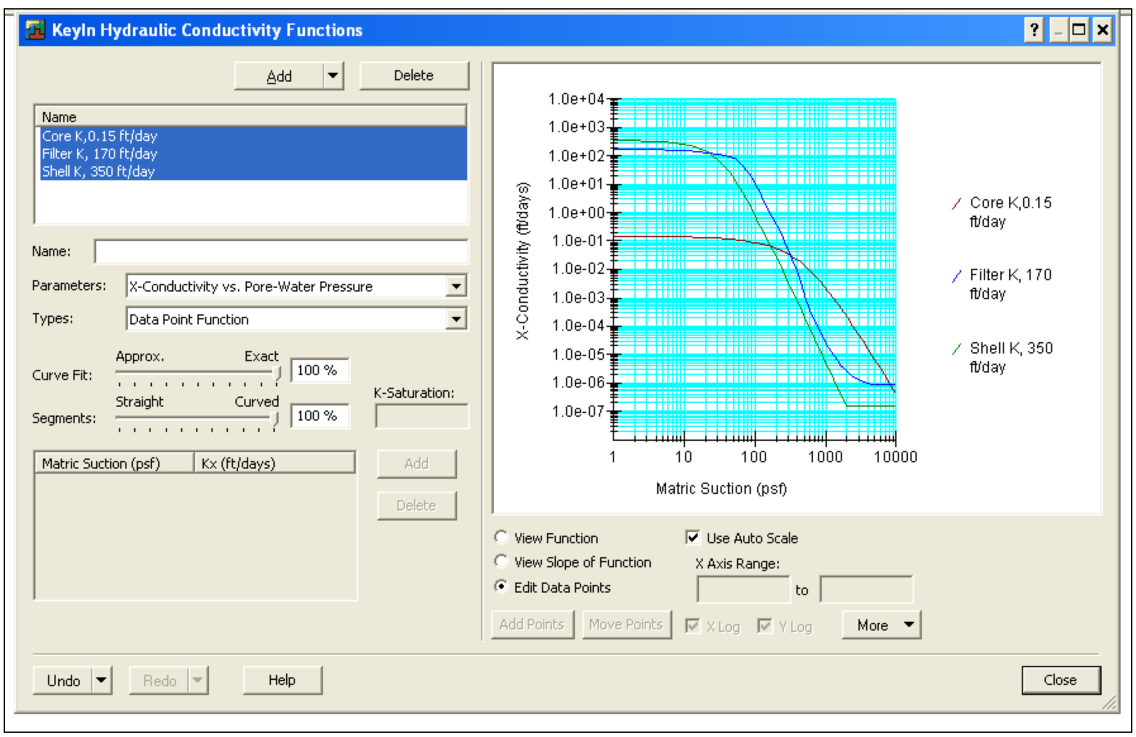


Figure C-3.3. Permeability functions – embankment materials.

III. Analysis

The seepage analyses include empty reservoir (initial condition), first filling, full reservoir, and rapid drawdown.

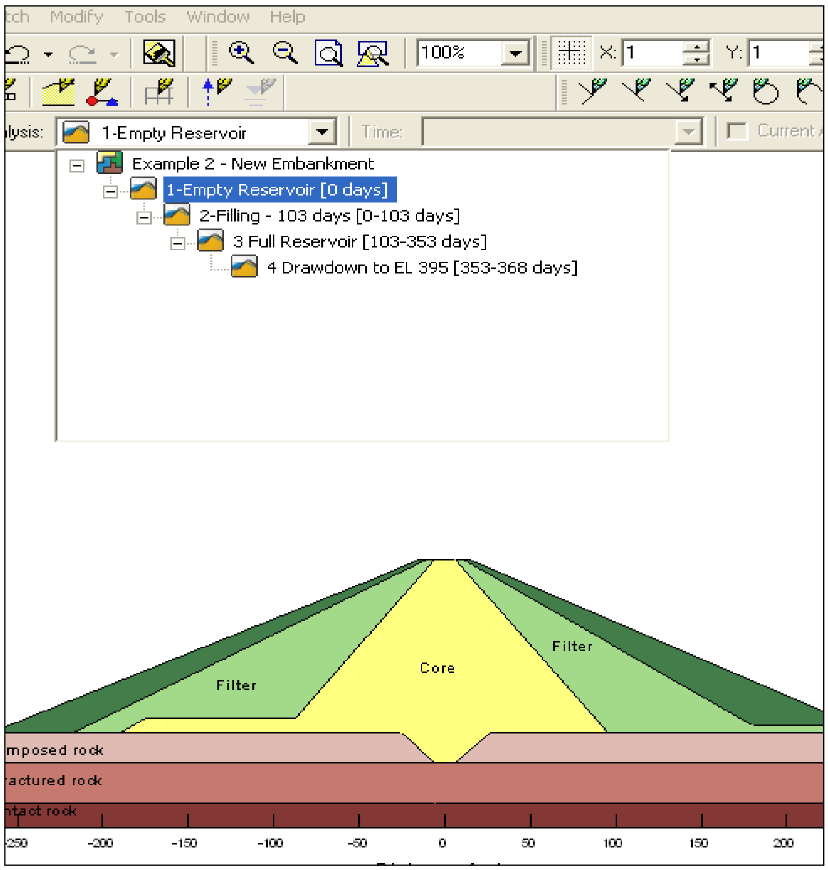


Figure C-3.4. Summary of four seepage analyses.

For the initial condition, prior to reservoir filling, the upstream and downstream ground surface should have zero pore pressure. Boundary conditions assigned to the upstream and downstream foundation surfaces are elevation head or a zero pressure head. The analysis type for the initial condition is steady state. This analysis resulted in the estimated phreatic line along the foundation ground surface as shown in figure C-3.5.

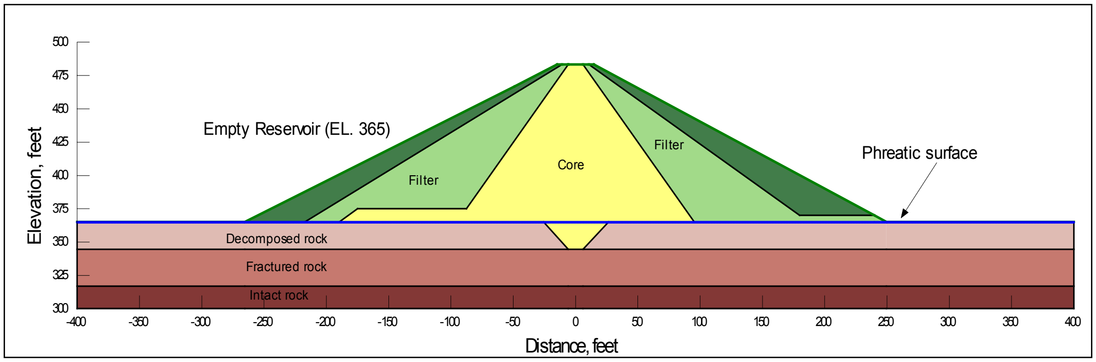


Figure C-3.5. Analysis result – empty reservoir.

The reservoir was then filled at a rate of 1.0 ft/d. Continuous filling from foundation surface elevation 365 to normal reservoir elevation 468 will require 103 days. The analysis type for the reservoir filling is a transient analysis. Figure C-3.6 shows the predicted phreatic line within the embankment after 103 days.

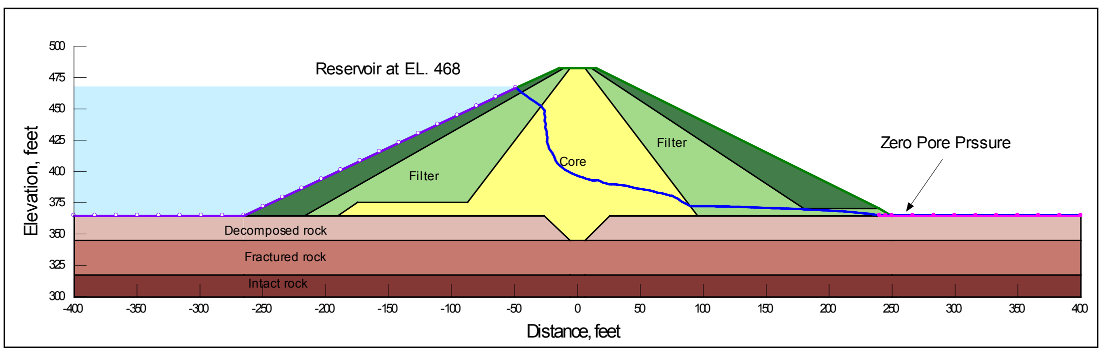


Figure C-3.6. Reservoir filling to EL. 468 – transient analysis.

After 250 days at reservoir elevation 468, the phreatic line in the core zone had advanced downstream as expected, and it was about at a steady state condition, as shown in figure C-3.7.

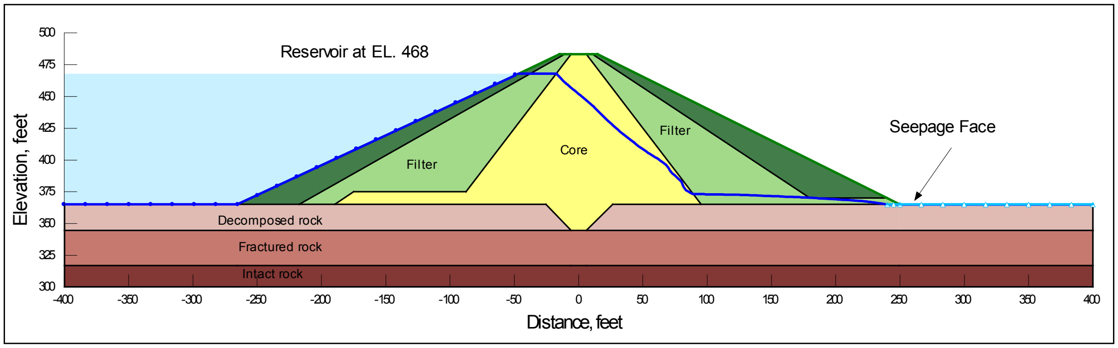


Figure C-3.7. Full reservoir EL. 468 for period of 250 days.

For a rapid drawdown condition, it was assumed that the reservoir would be evacuated down to elevation 393 (a total drawdown of 75 feet) within 15 days. A rapid drawdown with a rate of 5 ft/d was performed. Figure C-3.8 shows the phreatic line after drawdown.

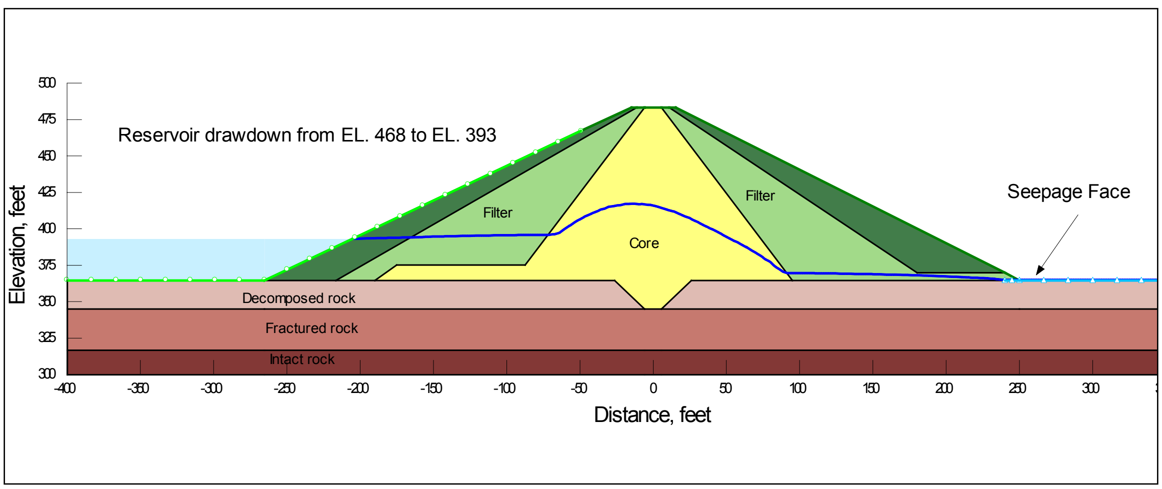


Figure C-3.8. Reservoir drawdown from elevation 468 to elevation 393.

Example 4 – Seepage with Embankment Core of Varying Thickness and Permeability

I. Introduction

The objective of this example is to examine steady state flow through a dam core for different values of saturated permeability of the core and for different thickness of the core zone.

The analysis includes:

1. Investigation of the seepage through a dam core when the saturated conductivity of the core is 10x, and 100x higher than the estimated baseline permeability of the existing core.

2. Comparison of the quantity of seepage for different assumed core widths.

II. Analysis Data

Figure C-4.1 shows the geometry of the existing dam. The dam has a central core zone flanked by filter zone on the upstream and downstream sides of the core. The outer zone is pervious gravelly shell.

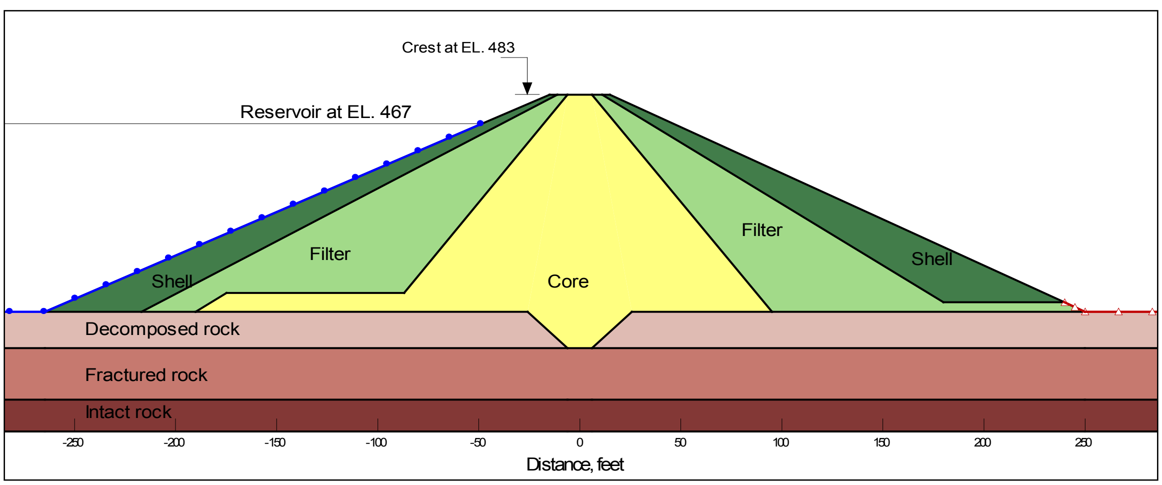


Figure C-4.1. Embankment dam with three zones of core, filter, and shell.

The boundary condition at the upstream end of the dam is a reservoir level at elevation 467. Seepage is assumed to exit into the downstream toe drain; therefore, the boundary condition at the downstream toe is assigned as a potential seepage face. All the embankment soils have a saturated and unsaturated function of permeability. Foundation layers were assumed to be below the water surface and, therefore, assigned only saturated permeabilities. The saturated permeability of the dam core zone was assumed to be about 2.0E-06 ft/s. The saturated permeabilities of the filter and shell were estimated at 2.0E-03 and 4.0E-03 ft/s, respectively.

III. Analysis Results

For core material having a saturated permeability of 2.0E-06 ft/s, a steady state seepage analysis estimates a seepage quantity through the dam and foundation of about 5.2E-04 ft^3/s/ft, as shown in figure C-4.2.

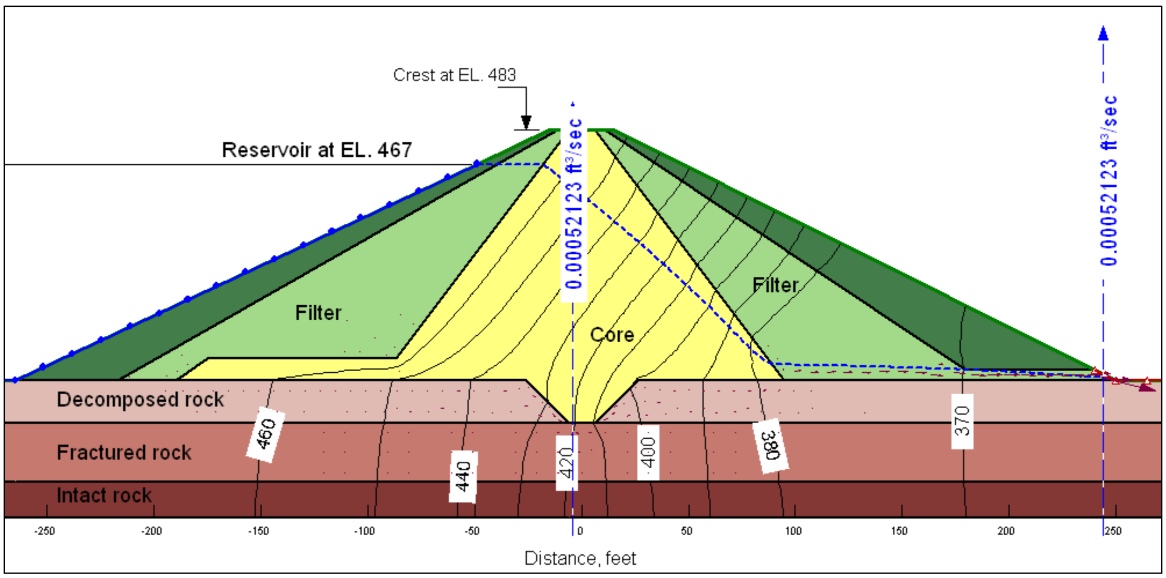


Figure C-4.2. Seepage through the dam with the baseline core permeability.

Assuming the saturated permeability of the core is 10 times higher than the baseline assumption, or 2.0E-05 ft/s, analysis results are shown in figure C-4.3.

Figure C-4.3 shows that phreatic line exits the downstream side of the core at a higher elevation, which is about elevation 379, compared to elevation 372.5 for the baseline case shown in figure C-4.2. As expected with more pervious core, the quantity of seepage is also increasing. The seepage quantity through the dam and foundation is estimated to be about 1.4E-03 ft^3/s/ft (about three times higher).

For the core having saturated permeability of 2.0E-04 ft/s, which is about 100 times higher than the existing core, seepage through the dam and foundation is estimated to be about 8.6E-03 ft^3/s/ft (more than 16 times higher).

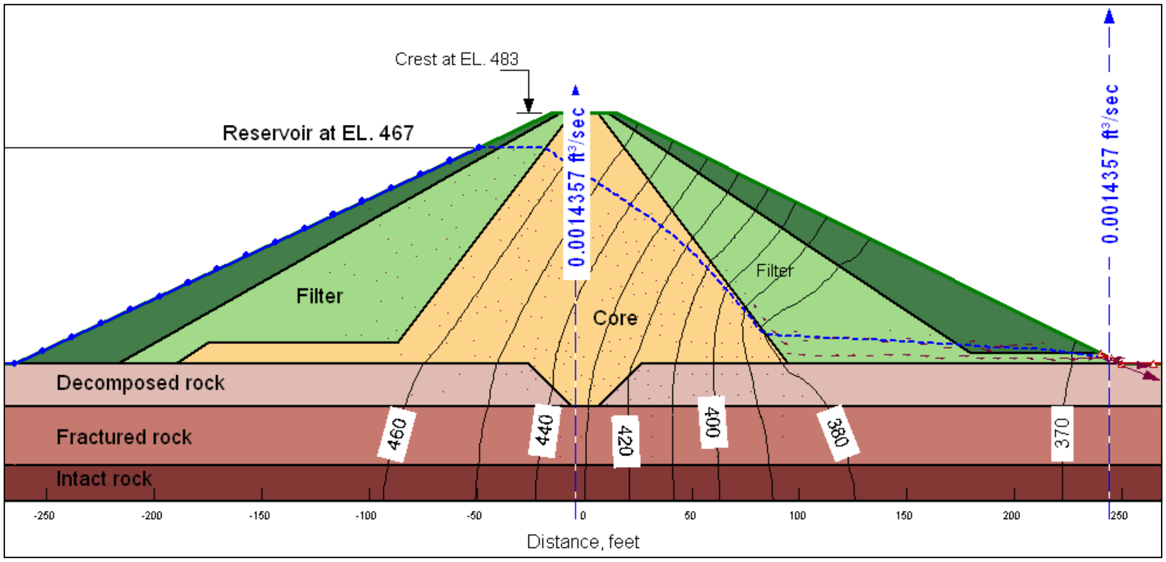


Figure C-4.3. Steady state seepage with 10 times more pervious core.

Figure C-4.4 shows that the potential seepage is estimated to exit the downstream slope of the embankment. The phreatic line exits the downstream side of the core at about elevation 409.

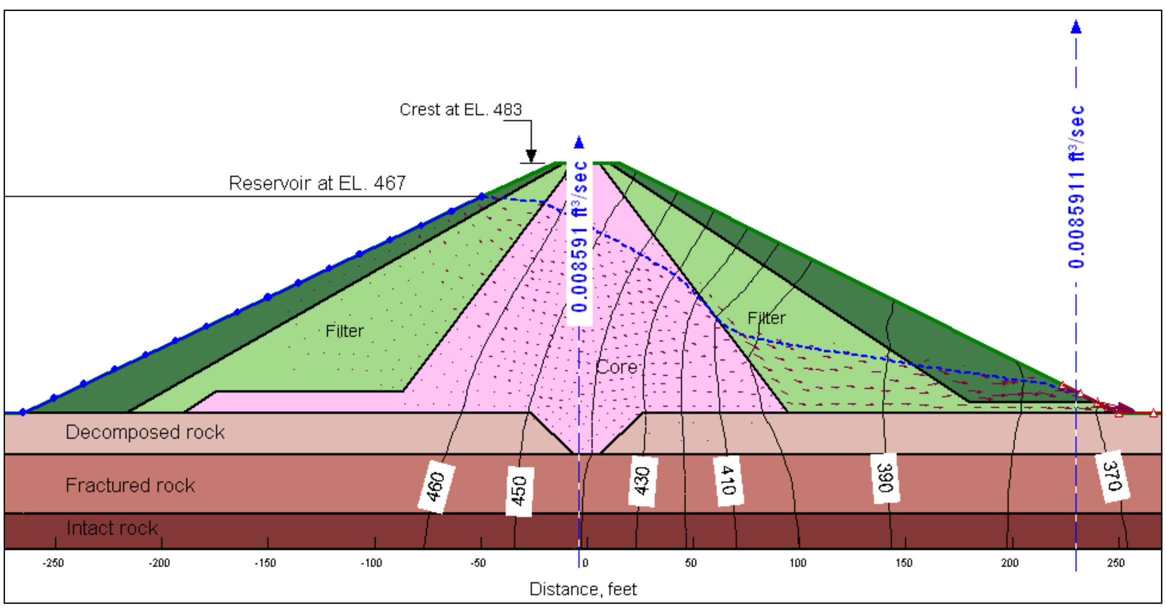


Figure C-4.4. Steady state seepage with 100 times more pervious core.

Next, the existing core was divided into three zones (upstream, middle, and downstream sections), as shown in the figure C-4.5 below.

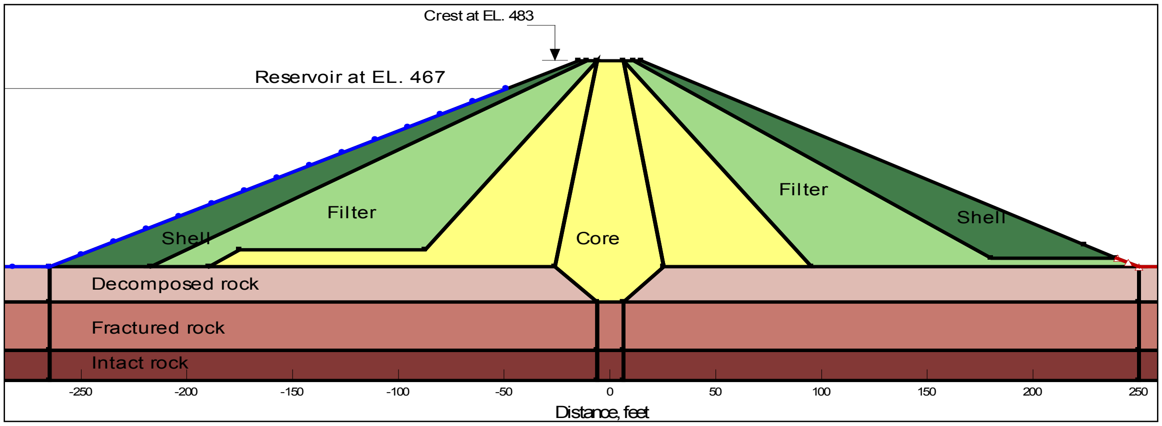


Figure C-4.5. Model of core as three zones.

The dam was analyzed with the following varying core zone thickness:

1. A thin central core, which is only the middle section, and the upstream and downstream sections assumed to have filter permeability.

2. The core zone is the upstream and middle sections.

3. The core zone is the middle and downstream sections.

Figures C-4.6 through C-4.8 show results of these three alternatives. As expected, the seepage was higher through a dam with a narrow core than the wider core.

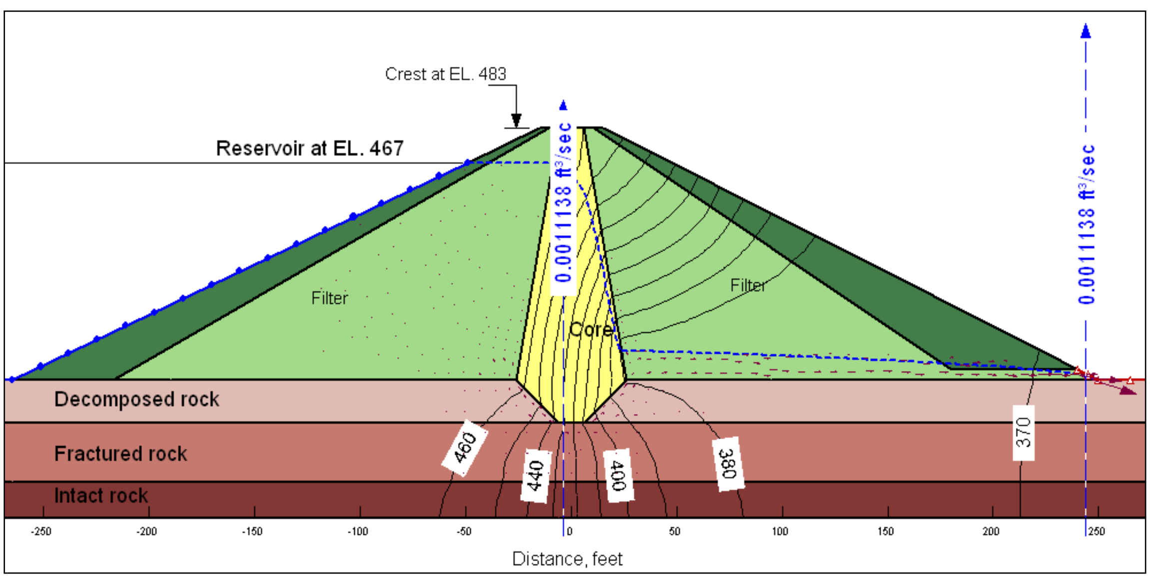


Figure C-4.6. Steady state seepage with thin central core.

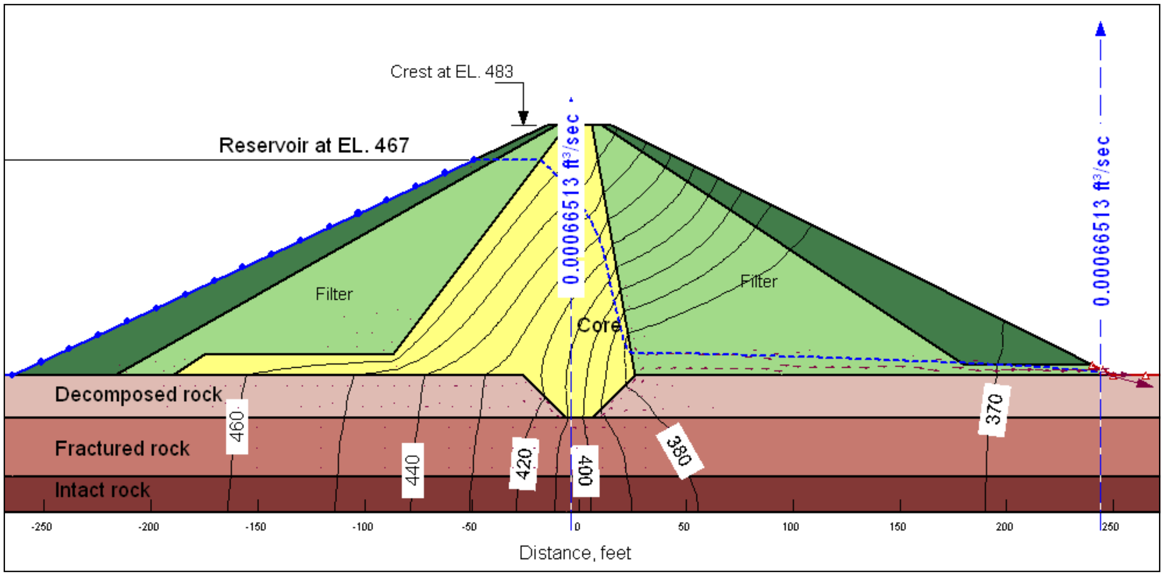


Figure C-4.7. Steady state seepage with thicker upstream core.

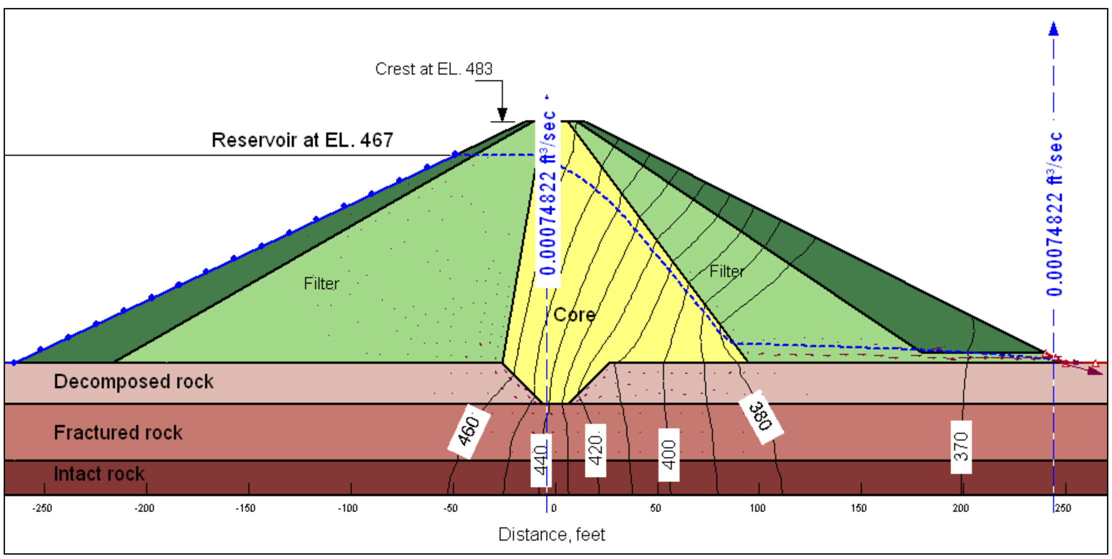


Figure C-4.8. Steady state seepage with thicker downstream core.

Table C-4.1 summarizes the results from these various models of core configurations and core permeabilities.

Table C-4.1. Estimated seepage quantity with varying core zone

Run	Permeability (ft/s)			Core configurations and assigned permeability			Seepage quantity
	Core	Upstream and Downstream Filter	Upstream and Downstream Shell	Upstream	Middle	Downstream	$ft^3/s/ft$
1	2.0E-06	2.0E-03	4.0E-03	Core	Core	Core	5.2E-04
2	2.0E-05	2.0E-03	4.0E-03	Core	Core	Core	1.4E-03
3	2.0E-04	2.0E-03	4.0E-03	Core	Core	Core	8.6E-03
4	2.0E-06	2.0E-03	4.0E-03	Filter	Core	Filter	1.1E-03
5	2.0E-06	2.0E-03	4.0E-03	Core	Core	Filter	6.7E-04
6	2.0E-06	2.0E-03	4.0E-03	Filter	Core	Core	7.5E-04

Example 5 – Various Models of an Embankment Drain

I. Introduction

This example presents how to model drains using different types of boundary conditions. This example models a steady state flow in a canal embankment with toe drains installed within its foundation. Three analyses were performed. The first case assumed no toe drain. With this assumption, and also depending on the permeability of the embankment and foundation materials, there is a chance that the estimated phreatic surface could exit at the canal slope surface.

In the second analysis, drains were placed at both left and right sides of the canal. The left drain is modeled as a hole (pipe) drain with a seepage face. The right drain is modeled as a free point drain with a zero pore pressure (P=0) boundary condition. The P=0 boundary condition is appropriate to use as long as no possibility exists that the pressures around the drain location will become negative. If this happens, the drain will become a source for water.

In the third analysis, the right drain is modeled as a free point drain with a Q=0 boundary condition. This boundary condition will also let water out if the pressures are positive or zero at the point; however, it will not let water back in if the soil has negative pressures.

II. Analysis Data

The canal embankment is about 2.5 meters high, and the canal invert width is about 5 meters. Water in the canal is about 0.5 meter deep. Ground water surface is about 1 meter below the ground surface. Figure C-5.1 shows the SEEP/W model.

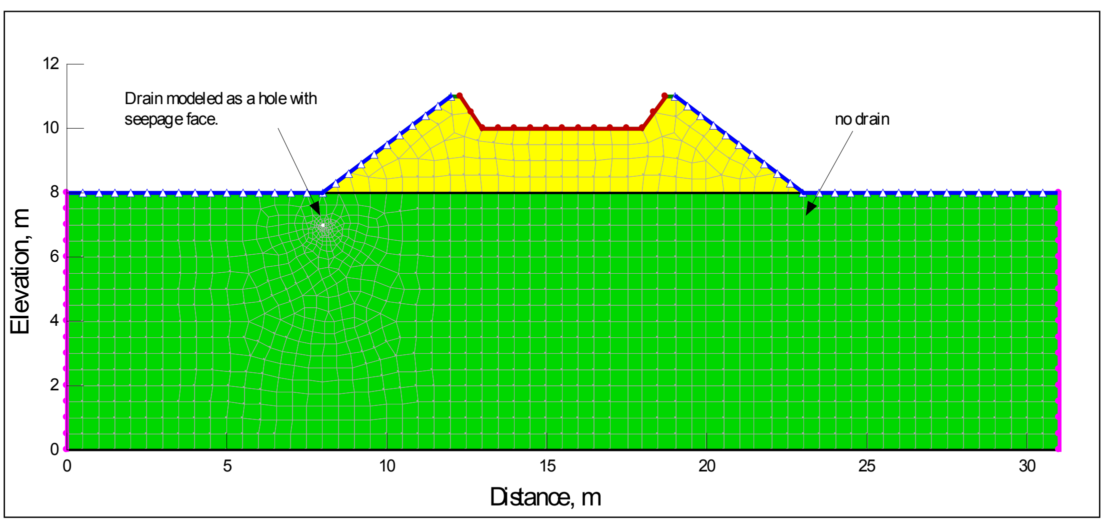


Figure C-5.1. Seepage model – canal with left and right toe drains.

The left drain was drawn as a circular opening region, as shown in figure C-5.2. The right drain was drawn as a free point region.

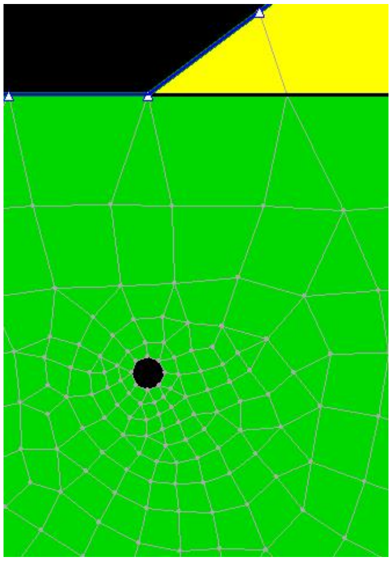


Figure C-5.2. Finite elements around the drain pipe.

Since all the analyses are for a steady state condition, volumetric water content is not required. The embankment material was assumed to have a saturated permeability of 1.0E-07 meters per second (m/s), and the foundation material permeability is about 5.0E-08 m/s. Figure C-5.3 shows the permeability functions of both embankment and foundation materials.

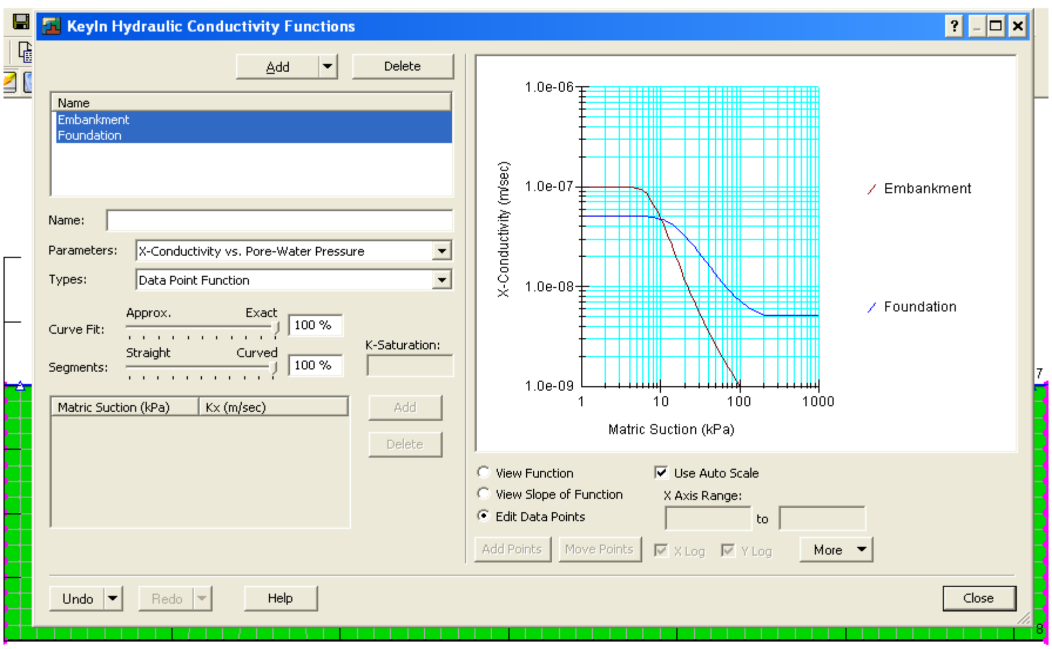


Figure C-5.3. Permeability functions of the embankment and foundation materials.

Boundary conditions at the left and right edges of the profile are ground water in the foundation, which is about 1 m below the foundation ground surface. With a datum at 8 m below the foundation ground surface, total head in the canal is about 10.5 m. The embankment slope surfaces and foundation ground surface are assigned as potential seepage surfaces.

III. Analysis Results

A. Case 1 – No Drainage Structures

With no drainage features and a foundation material less pervious than the embankment material, the phreatic line was estimated to exit the slope surfaces, as shown in figure C-5.4.

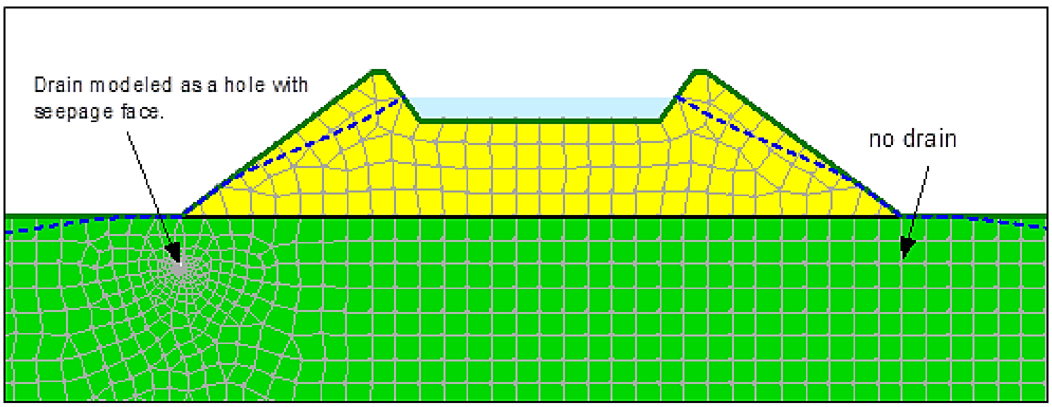


Figure C-5.4. Analysis results of Case 1 – no drainage structure.

B. Case 2 – Left Drain as Seepage Face; Right Drain Has Boundary Condition of P~0

The boundary condition assigned to the left drain is the seepage face, and for the right drain is assigned a zero pore pressure (P=0). Figures C-5.5 and C-5.6 show input configurations.

Figure C-5.7 shows the results of the Case 2 analysis. As shown in the figure, seepage no longer exits the downstream face of the embankment.

Figure C-5.5. Case 2 – boundary conditions for the left and right drains.

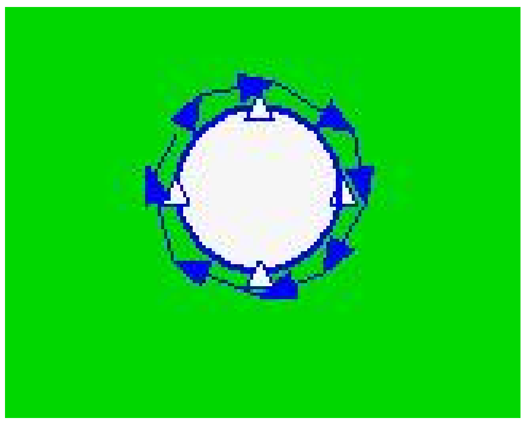


Figure C-5.6. Case 2 – left drain – seepage face boundary condition.

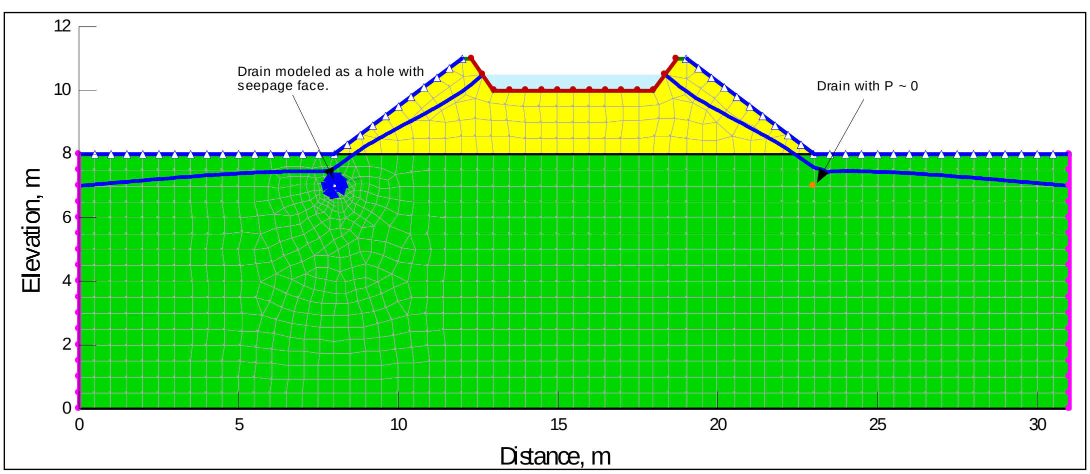


Figure C-5.7. Analysis results of Case 2 – toe drains on each side.

With drainage placed on both sides of the canal, phreatic lines were lowered. The quantity of seepage through the left drain and the right drain is the same. From "View Result Information," shown on figure C-5.8, it can be seen that the total flux (seepage quantity) through the right drain is about 1.6E-07 m^3/s. This is the approximate same value of seepage flow that is entering the left drain, as shown on figure C-5.9.

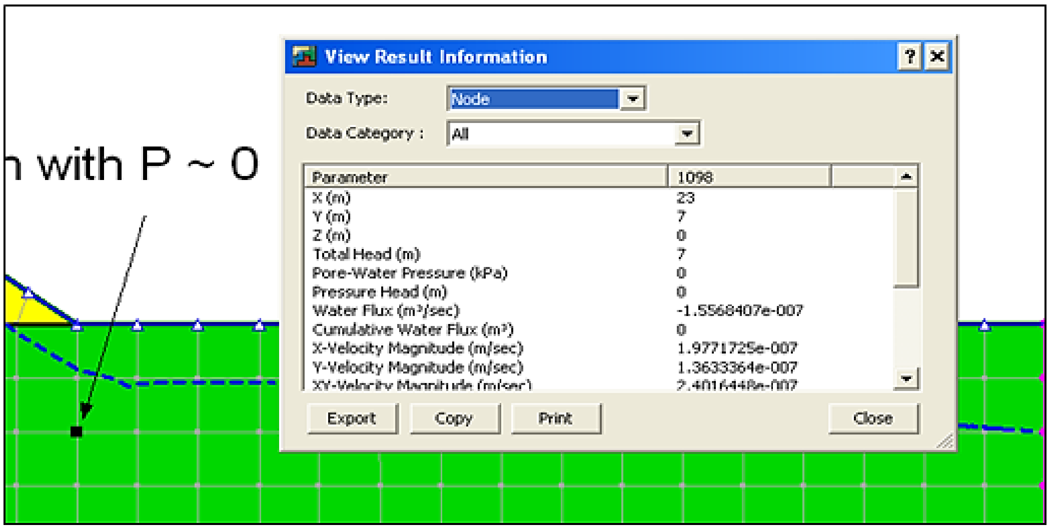


Figure C-5.8. Summary of results at the right drain node.

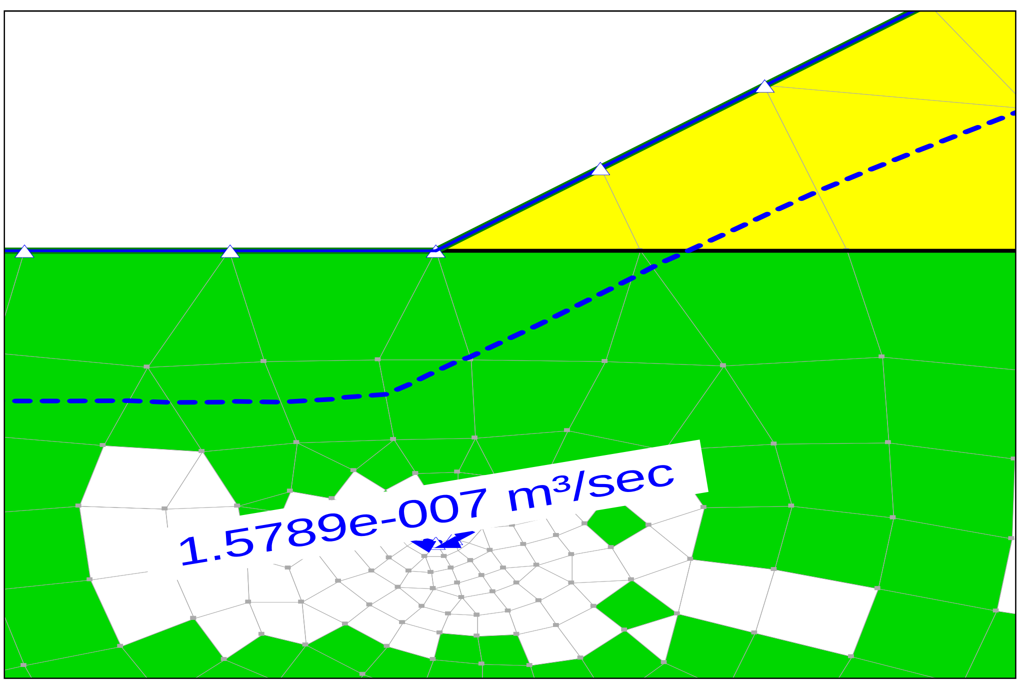


Figure C-5.9. Summary of results at the circular opening (left drain).

C. Case 3 – Boundary Condition of Q=0 for the Right Drain

The boundary condition for the left drain is the same as Case 2, while the right drain was assigned a Q=0 boundary condition, as shown on figure C-5.10, which is a seepage face review.

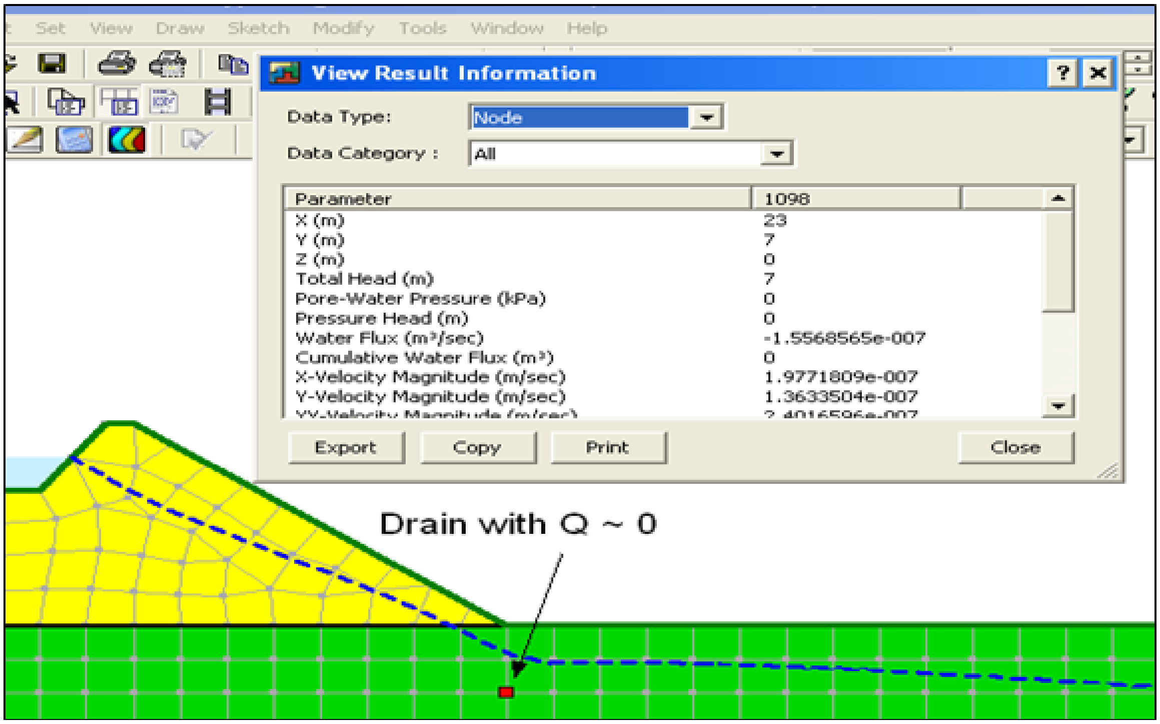


Figure C-5.10. Summary of results for right drain boundary.

The nodal flow is computed as -1.6 e-7 m^3/s, which agrees with Case 2.

Example 6 – Relief Wells at the Downstream Toe of an Embankment

I. Introduction

This example is from the GeoStudio Web site (www.geo-slope.com). This type of analysis could be used to evaluate the required number of wells needed to control seepage at the downstream toe of a dam. This example explores the relative effect of well spacing on the seepage and pore pressures. The option in SEEP/W to analyze this problem is a "plan view" option. This type of analysis uses a simplifying assumption that the embankment acts as a confining structure and the foundation soils act as an "aquifer." Since the seepage was assumed to be flowing beneath the structure, foundation soil was assumed to have only a saturated permeability; therefore, no permeability function is required.

II. Analysis Data

As shown in figure C-6.1, an embankment is founded on a 10-meter-thick foundation layer. The reservoir level (total head) is 13 meters above the datum, which is located at the bottom of the foundation layer. The "far-field" total head downslope of the embankment is defined as 7 meters, which is the ambient ground water condition.

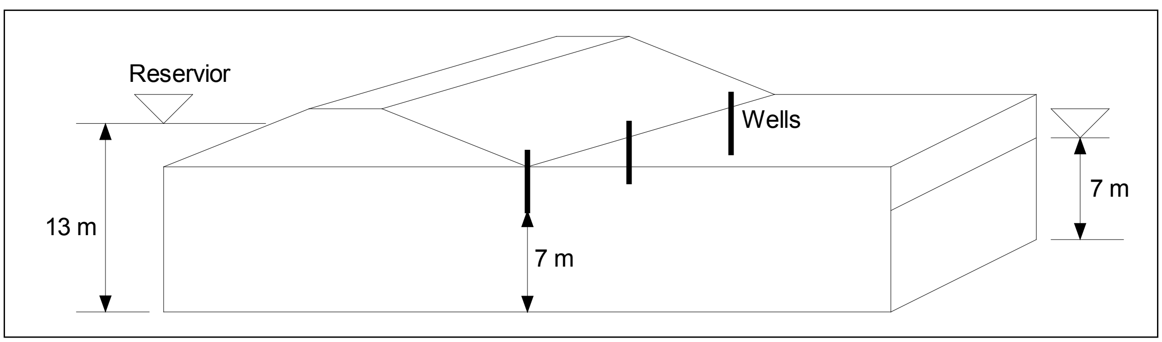


Figure C-6.1. Conceptual model.

For the purpose of this example, it is assumed that pumping maintains the water level at 7 meters in the relief wells (the same elevation as the water in the seepage outlet). The foundation permeability was assumed to be about 1 meter per day (m/d).

Assuming the longitudinal length of the modified embankment is about 50 meters, the upstream length (x-coordinates) is between 0 and 50 meters (points 1 and 2 in figure C-6.3). The far field boundary is about 50 meters from the upstream point, as shown the x-coordinates value of point 3. The thickness of foundation of

10 meters is shown as a z-coordinate value. Figures C-6.2 and C-6.3 illustrate the input parameters used for the model.

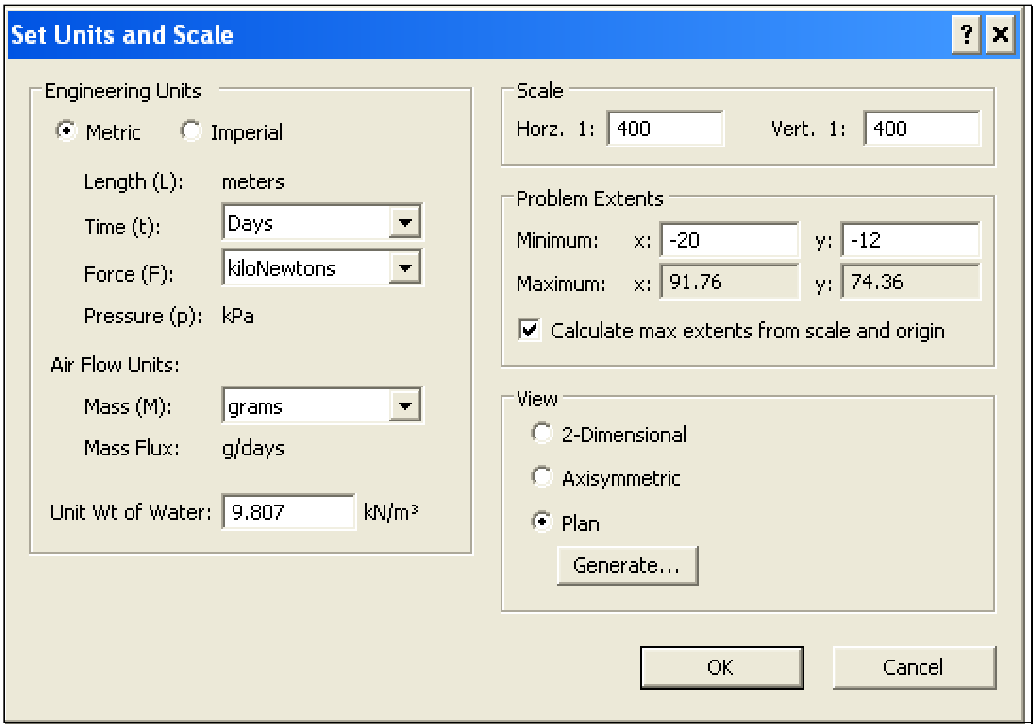


Figure C-6.2. The plan view option, selected using the set units and scale command.

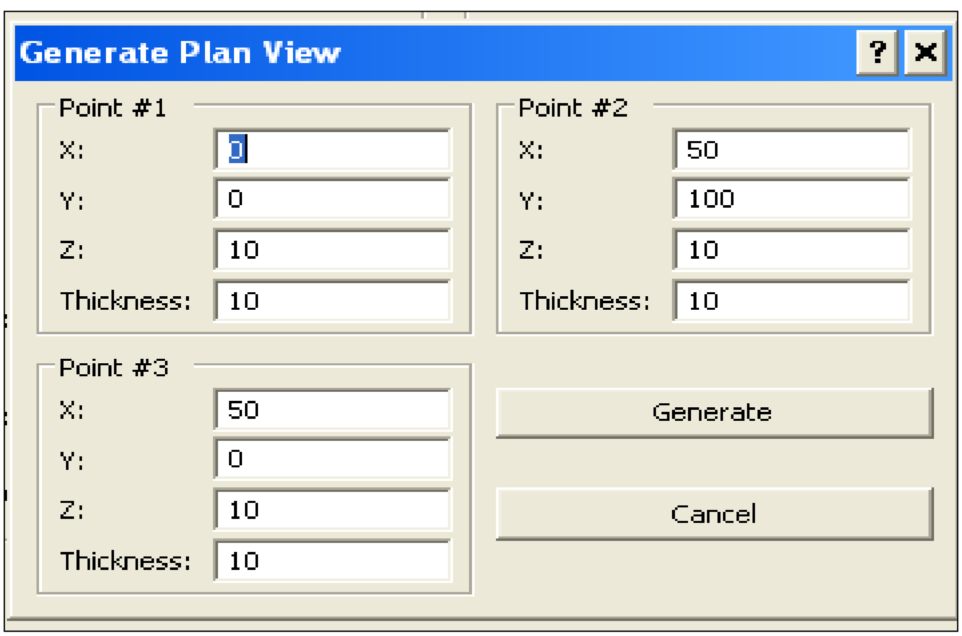


Figure C-6.3. Plan view dialog box for mesh thickness generation.

Figure C-6.4 shows the plan view of the SEEP/W configuration.

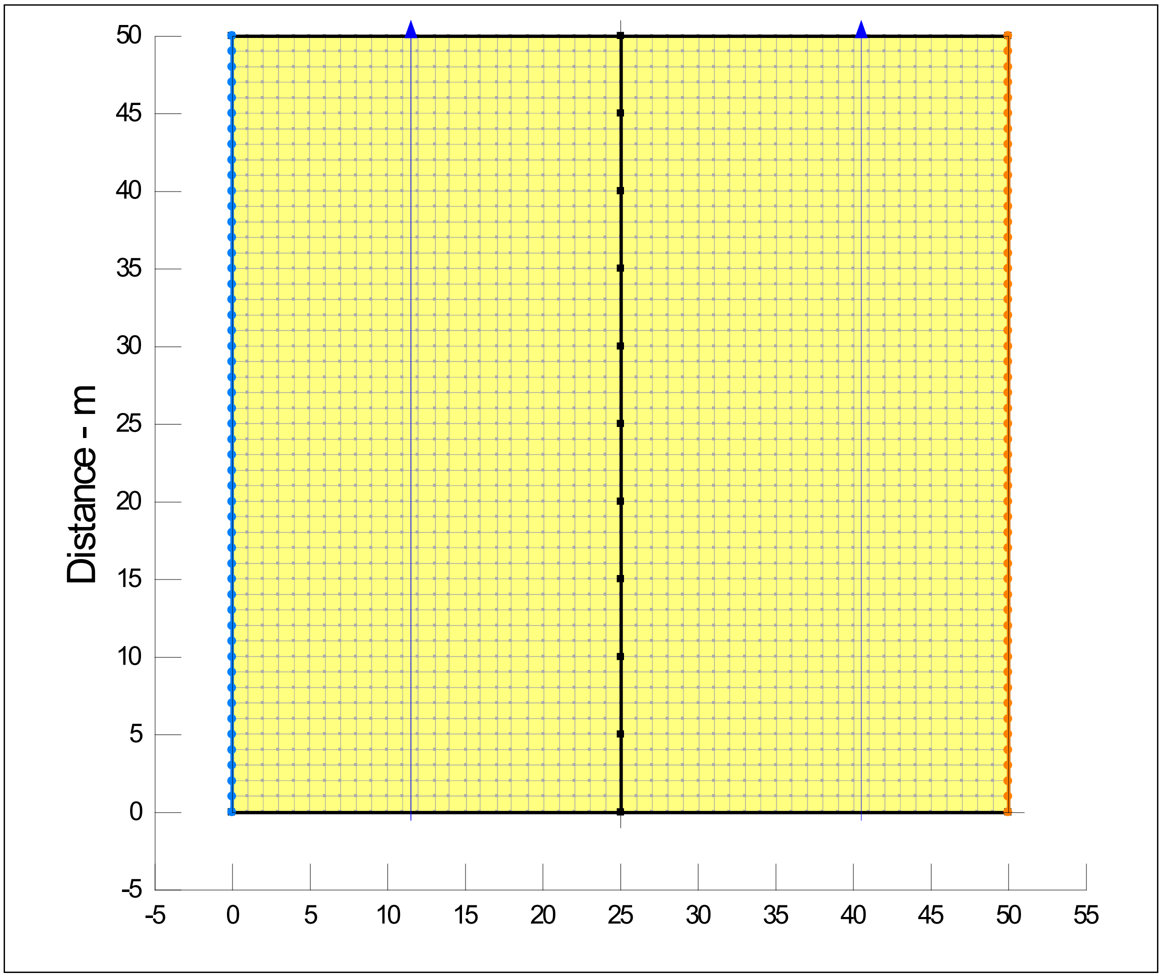


Figure C-6.4. SEEP/W profile – plan view option.

The type of analysis to be used is a steady state analysis. The initial condition is before installation of wells. The following analysis is for wells with spacing of 50, 25, 10, and 5 meters.

III. Analysis Results

For the initial condition with no wells, the seepage through the foundation was estimated to be about 60 cubic meters per day (m^3/d), as shown in figure C-6.5. The predicted seepage flows at two locations, beneath the crest and downstream of the location of the proposed wells (downstream toe), are the same. Total head at the proposed location of the wells is at elevation 10 meters.

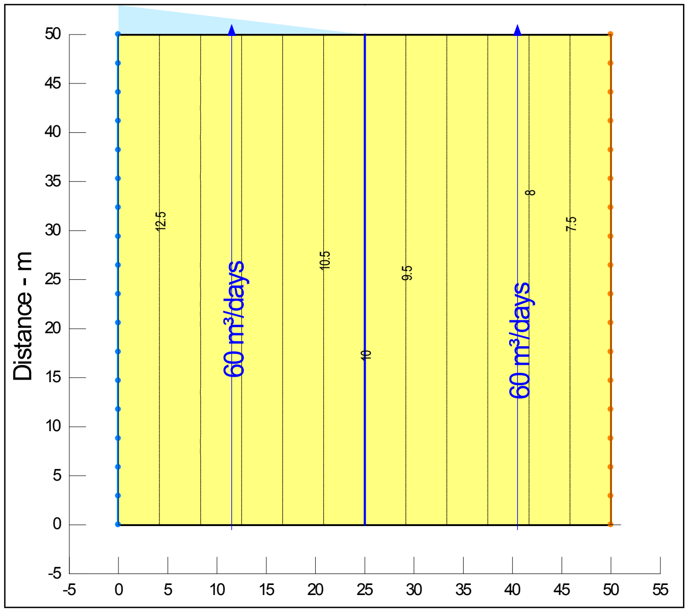


Figure C-6.5. Estimated seepage and total head – no wells installed.

With installation of wells, the total head at the well locations is lowered, and seepage downstream of the well locations is also less. The flow through the foundation area beneath the crest is increasing with increasing number of wells, while flows at the downstream toe are decreasing, as shown in figures C-6.6 through C-6.8.

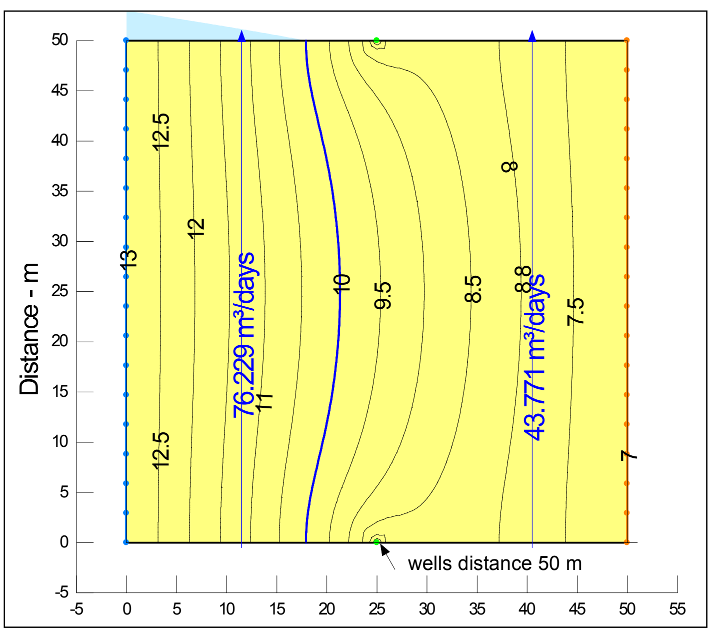


Figure C-6.6. Estimated seepage and total head – two wells installed (50-meter spacing).

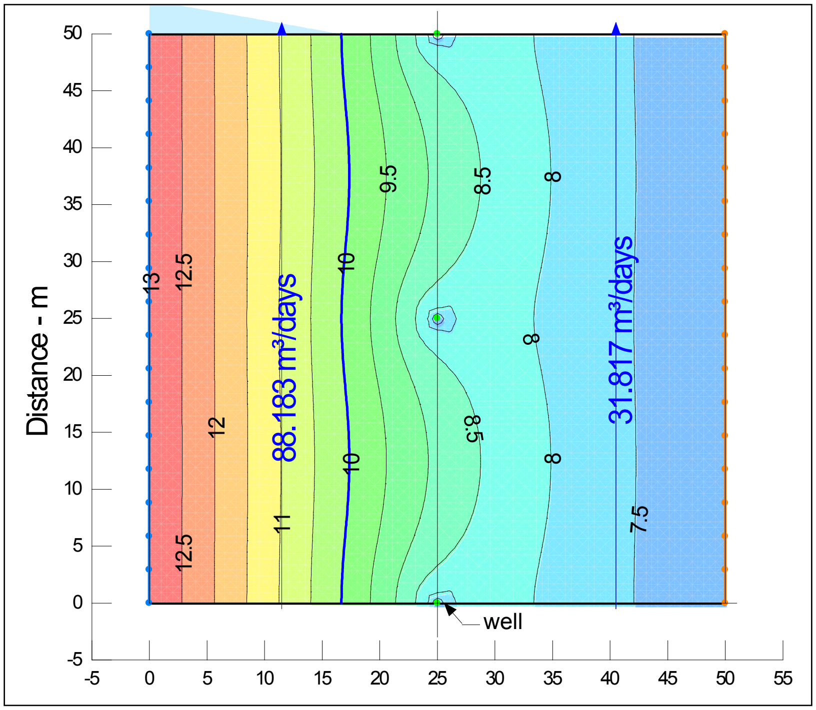


Figure C-6.7. Estimated seepage and total head – three wells installed (25-meter spacing).

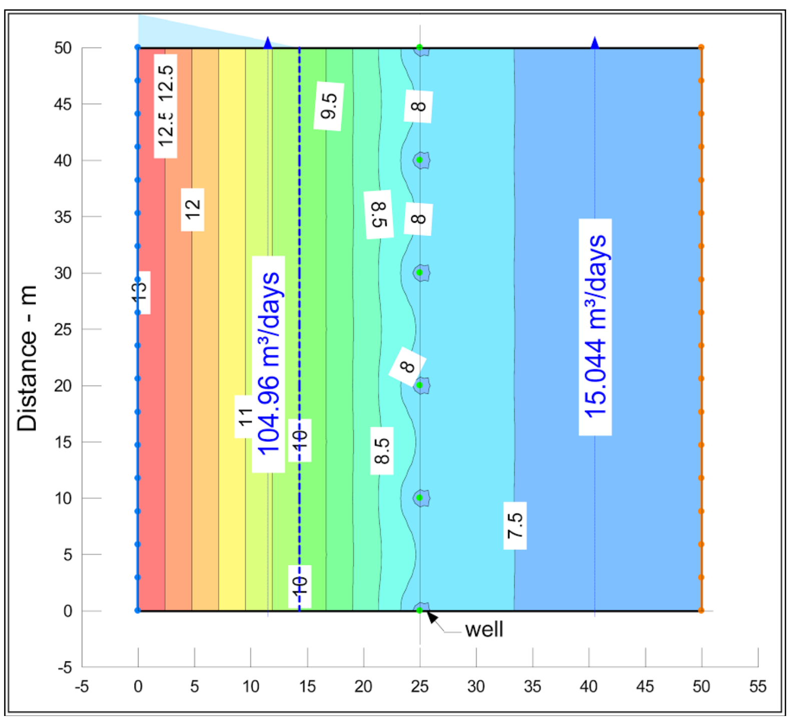


Figure C-6.8. Estimated seepage and total head – six wells installed (10-meter spacing).

A final configuration that was modeled was a continuous trench excavated along the downstream toe, reflecting a continuous drainage feature. Figure C-6.9 shows the results of that model. The drainage trench essentially captures all but a very small portion of the seepage (note small flows at downstream end of model).

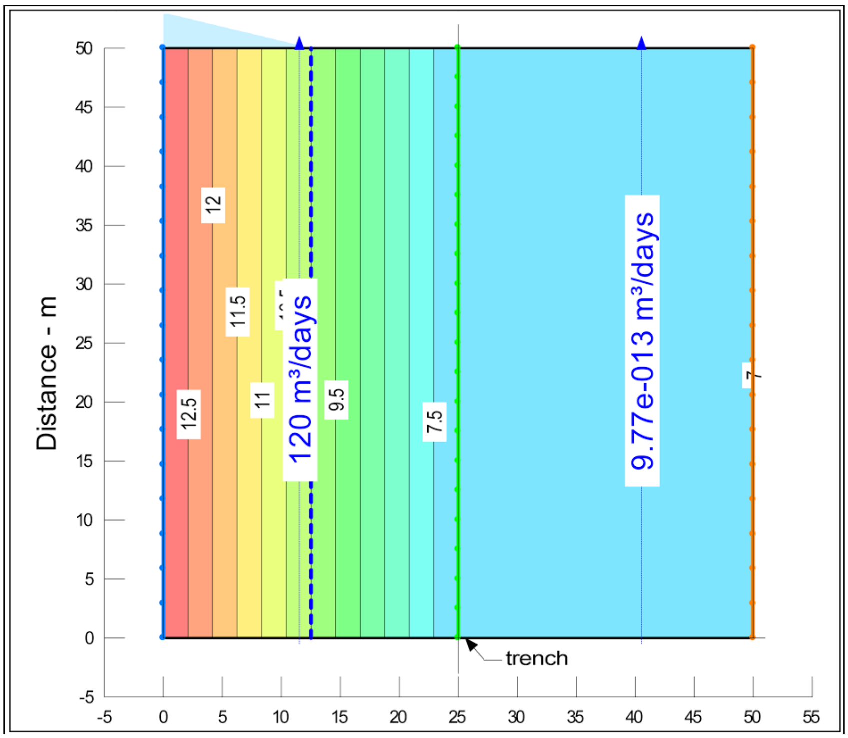


Figure C-6.9. Estimated seepage and total head – continuous trench.

Table C-6.1 shows the results of all model simulations.

Table C-6.1. Estimated Seepage and Total Head for All Models

No.	Well Number (well distance, meters)	Seepage quantity (m^3/d)		Maximum Total Head (meters)
		Upstream of Wells	Downstream of Wells	
1	No well	60	60	10
2	2 (50 meters)	76.23	43.77	9.5
3	3 (25 meters)	88.18	31.82	8.8
4	6 (10 meters)	104.96	15.04	7.8
5	11 (5 meters)	113.44	6.56	7.1
6	Continuous trench	120	9.77E-13	Close to 7.0

IV. Conclusions

The plan view option in SEEP/W can be used to explore the relative effect of well spacing on seepage flows and pore pressures. The term “relative” is used here because the plan view simulation is not a true three-dimensional analysis.

The Geo-slope user’s guide stated that the results of the plan view analysis are best viewed as relative values for various spacing. Actual seepage quantities would be better represented by a conventional two-dimensional analysis.