Filters for Embankment Dams

Best Practices for Design and Construction

October 2011

FEMA

Filters for Embankment Dams
Best Practices for Design and Construction

Federal Emergency Management Agency

Contents

Figures and Tables

Figures

Tables

Preface

The Federal Emergency Management Agency's (FEMA) National Dam Safety Program sponsored development of this document in conjunction with the Association of State Dam Safety Officials, Bureau of Reclamation (Reclamation), Federal Energy Regulatory Commission, Natural Resources Conservation Service (NRCS), and U.S. Army Corps of Engineers (USACE).

The primary authors of this document (and their affiliations during authorship) were Mark W. Pabst, P.E. (Reclamation), Danny K. McCook, P.E. (NRCS), James R. Talbot, P.E. (NRCS, retired), David P. Hammer, P.E. (USACE, retired), Noah D. Vroman, P.E. (USACE), and Landris T. Lee, P.E. (USACE-ERDC). The technical editor for this document was Sharon Leffel (Reclamation), and the illustrator was Cynthia Gray (Reclamation).

A number of engineers and technicians provided input in preparation of this document, and the authors appreciate their efforts. The authors extend their appreciation to the following individuals for graciously providing peer reviews, information, and/or permission to use their materials in this publication:

John W. France, P.E. (URS)
William O. Engemoen, P.E. (Reclamation)
Daniel Wade Anderson, P.E. (USACE)
David E. Kleiner, P.E. (MWH Americas)
John Cyganiewicz, P.E. (Reclamation, retired)
Robin Fell (University of New South Wales, Emeritus)
Thomas L. Brandon, P.E. (Virginia Tech University)
Michael D. Ramsbotham, P.E. (USACE)
Mia P. Kannik, P.E. (Ohio Department of Natural Resources)
Chris B. Groves, P.E. (Shannon and Wilson)
Mark R. Haynes, P.E. (Colorado Division of Water Resources)
Robert P. Chapuis, (École Polytechnique de Montréal)
Thomas I. Roberts. P.E. (Virginia Division of Dam Safety)
Chi Fai Wan (Black and Veatch, Australia Pty Ltd)
Edward G. McGregor, P.E. (USACE, retired)

Michael K. Sharp, P.E. (USACE-ERDC)
Richard W. Peterson, P.E. (USACE-ERDC)

The material presented in this document has been prepared in accordance with recognized engineering practices. The guidance in this document should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the material contained herein is not intended as representation or warranty on the part of individuals or agencies involved, or any other person named herein, that this information is suitable for any general or particular use, or promises freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability from such use.

Any use of trade names and trademarks in this document is for descriptive purposes only and does not constitute endorsement. The information contained herein regarding commercial products or firms may not be used for advertising or promotional purposes and is not to be construed as an endorsement of any product or firm.

COL Gary E. Johnston was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

Introduction

Filters in embankment dams are composed of specifically-designed entities (zones) of coarser-grained soils placed at specifically-targeted locations within or adjacent to the dam structure. Filters are designed and constructed to achieve specific goals such as preventing internal soil movement and controlling drainage, and are typically installed during new dam construction. Filters have also been added to existing dams to meet specific requirements.

This document provides procedures and guidance for best practices concerning embankment dam filter design and construction, and represents an effort to collect and disseminate current information and experience having a technical consensus. Currently available information was reviewed, and when detailed documentation existed, it was cited to avoid duplicating available materials. The authors have strived not to reproduce information that was readily accessible in the public domain and attempted to condense and summarize the vast body of existing information, provide a clear and concise synopsis of this information, and present a recommended course of action.

The authors acknowledge that there are variations in the procedures and standards for filter design and construction. They focused on what they judged to be the "best practice" and included that judgment in this document. Therefore, this document may be different than some of the various participating agencies' own policies.

This document is intended for use by personnel familiar with embankment dams, such as designers, inspectors, construction oversight personnel, and dam safety engineers. The users of this document are cautioned that sound engineering judgment should always be applied when using references. The authors have strived to avoid referencing any material that is considered outdated for use in modern designs. However, the user should be aware that certain portions of references cited may have become outdated in regards to design and construction aspects and/or philosophies. While these references still may contain valuable information, users should not automatically assume that the entire reference is suitable for design and construction purposes.

If filters are not designed and constructed correctly, embankment dams will have an increased probability of failure, which endangers the public. The particular design requirements and site conditions of each embankment dam are unique, and as such, no single publication can cover all of the requirements and conditions that can be encountered during design and construction. Therefore, it is critically important that embankment dam filters be designed by engineers experienced with all aspects of the design and construction of embankment dams.

Terminology

Through the decades, a number of terms have been used in association with dams and filters. Some, due to their historical context, may be confusing. This section will present some of the terminology and describe the nomenclature used throughout this manual. The Glossary at the end of this manual is also explanatory.

Dam hazard classification

Embankment dams, regardless of their size, create a hazard potential from the stored energy of the water they impound. Examples, such as Kelley Barnes Dam, which failed suddenly in 1977, show the destructive power of water when it is released suddenly from behind even a small embankment dam. This embankment dam was less than about 40 ft high and about 400 ft long, but when it failed, it released water downstream at an estimated flow rate of over 24,000 cubic feet per second, killing 39 people.

The hazard potential of an embankment dam is based on the consequences of failure rather than its structural integrity, and the hazard potential classification does not address the condition of the dam (i.e., safety, structural integrity, or flood routing capacity). Hazard potential classifications are assigned based on the dam's potential for causing downstream damage but say nothing about the safety or condition of the structure. An embankment dam might be classified as having a low hazard potential based on the low impact that a failure would have on the downstream area but yet have a high probability of failure if it were in very poor condition.

The three hazard potential classification levels used in this document are low, significant, and high as defined in FEMA 333:

- *Low hazard potential*—Embankment dams assigned the low hazard potential classification are those in which failure or mis-operation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.
- *Significant hazard potential*—Embankment dams assigned the significant hazard potential classification are dams in which failure or mis-operation does not result in loss of human life but can sustain economic or environmental damage as well as many other types of property and infrastructure damage. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas, but could be located in areas with population and significant infrastructure.
- *High hazard potential*—Embankment dams assigned the high hazard potential classification are those in which failure or mis-operation will probably cause loss of human life.

Often, low hazard embankment dams are small structures (height or reservoir volume). The term "small embankment dam" does not have a single widely accepted definition. Some regulations may consider a 25-ft-high embankment dam to be the largest dam in the small dam category, and others may consider this to be the smallest dam in the large dam category. For example, the International Commission on Large Dams (ICOLD 1994) defines large embankment dams as being more than about 50 ft high. The guidance in this document is considered to be technically valid without regard to either the physical size or hazard potential classification of an embankment dam.

Filter versus drain

Historically, the terms *filter* and *drain* have held different meanings by different authors, and their use as both nouns and verbs has led to some confusion. Filter material, when designed using the guidance in this manual, provides both particle retention and drainage in embankment dams. Therefore, a single material can retain or *filter* particle movement from a base soil and have sufficient permeability to act as a *drain*. Since the designed material performs both function s, the terms have become interchangeable, especially in relation to where the material is used in the embankment cross section. This has led to some authors using the word drain for a filter and vice versa. Others have chosen to combine the terms into *filter/drain*, *filter-drain*, and *filter and drain*.

Typically, the distinction between these terms can be made based on the stage (sequential pattern or interval). As described in the manual, a first-stage filter protects the base soil (core), and its primary function is particle retention. In many instances, a second-stage material will also be used, and its primary function is to provide drainage. While both materials meet particle retention and drainage criteria, the emphasis of the first stage is on particle retention, and the emphasis of the second stage is on drainage. In accordance with this philosophy, this manual will use the term *filter* in the context of embankment zones as the first-stage material. In a similar manner, the term *drain* will be used for zones that function as second-stage material. As an example, for a two stage chimney, the first stage would be the *chimney filter* and the second stage would be the *chimney drain*. For cases in which both stages are present, the term *filter/drain* will be used.

Grain size distribution plots

The soil particle size gradation graph (also called the grain size cumulative distribution curve) is the primary filter design tool used in this manual. This plot is the physical representation of the dam's base soil material and filter material, and its proper usage and interpretation must be emphasized. For example, the conventional geotechnical method for plotting the cumulative grain size distribution curve reverses the x-axis numerical scale (particle size in millimeters). Instead of plotting the x-axis data in ascending order from left to right, grain size distribution curves are traditionally plotted with the x-axis data in descending order left to right.

The reader is encouraged to note this traditional plotting convention and to be aware that some grain size distribution curves may not be presented in the traditional descending scale fashion. The curves may appear to be horizontally flipped from the traditional fashion. This plotting inconsistency may lead to confusion when establishing filter material bandwidth (maximum and minimum particle sizes), discussed later in this manual.

Particle size gradation

The most prominent component of filter design is the soil particle size gradation (i.e., grain size cumulative distribution) of the base soil (i.e., the soil to be protected, or the dam core) and the filter soil (i.e., the soil providing the protection). It is a common practice to describe a soil based on its grain size distribution, or gradation. Since soils behave differently, in an engineering sense, if they are of all one particle size or if they have a wide range of sizes, terms came into being to describe these two different soil gradation signatures. Since most filter soils are coarser-grained, similar to concrete mix aggregates, it was recognized during concrete mix design that aggregates containing roughly equal amounts of sand and gravel made for a stronger and more economical product than aggregates that were only composed of sand. Therefore, aggregate gradations that had roughly equal parts sand and gravel were called *well graded* since they performed well in concrete. In a similar manner, gradations that only included sand sizes were termed *poorly graded* due the poor performance of that mix design. While broadly (well) graded soils are acceptable in some filter applications, it should not be concluded that they are superior to more uniformly (poorly) graded soils. Uniformly (poorly) graded soils are preferred for use in two-stage designs such as toe drains, and it should not be inferred that they are "poor" or unacceptable for use.

To help alleviate this confusion, new terms were introduced that were more generic to the shape of the gradation curve and did not focus on the performance of a particular gradation. Gradations that included many soil types, and when viewed on the gradation plot had a broad appearance, were named *broadly graded*. On the other hand, a gradation of a single soil type that appeared to be narrow on the gradation chart was named *narrowly graded*. Since these narrow gradations were also uniform in their distribution, the term *uniformly graded* was also used. Therefore, the following terms are synonymous:

Narrowly graded	=	Uniformly graded	=	Poorly graded
Broadly graded	=	Widely graded	=	Well graded

When using the standardized Unified Soil Classification System (USCS) gradation methodology (hereinafter utilized exclusively), the distinction between well and poorly graded soils is made via the coefficient of uniformity (Cu) and the coefficient of curvature (Cc) parameters, where

$$\text{Coefficient of uniformity, } Cu = D_{60}/D_{10}$$

In this manual, the standard coefficient of curvature symbol (Cc) is replaced by the symbol (Cz) to avoid confusion with the standard compression index symbol (Cc).

$$\text{Coefficient of curvature, } Cz = Cc = D_{30}^2 / (D_{60} * D_{10})$$

where D_{60}, D_{30}, and D_{10} are the particle diameters corresponding to 60%, 30%, and 10% finer on the particle grain size cumulative distribution curve, respectively.

Well (broadly) graded soils are defined in the USCS as:

$$Cu \geq 4 \text{ and } (1 < Cz \leq 3), \text{ (i.e., Cz is between 1 and 3, inclusive)}$$

Poorly (uniformly) graded soils are defined by:

$$Cu < 4 \text{ and/or } (Cz < 1 \text{ or } Cz > 3),$$
$$\text{(i.e., Cz is not in the interval between 1 and 3)}$$

Figure 1 is a plot that illustrates the descriptive gradations.

Two other terms used to describe the gradation of a soil are *gap graded* and *skip graded*. These terms essentially mean the same thing and describe that condition when a range of grain sizes are missing from a gradation. The terms came into use upon observation of the gradation test where some sieves would have little or no soil particles retained. In other words, a range of sieves were *skipped*, there was a *gap* in the gradation data, or (most importantly) there was an *absence* of certain particle sizes. Figure 2 is a gradation plot that illustrates this soil type. This manual will use the term *gap graded* for these types of soils. Note that gap graded soils can be internally unstable meaning that finer particles in the soil matrix can be removed through the constrictions between the coarser particles in the soil matrix during water flow.

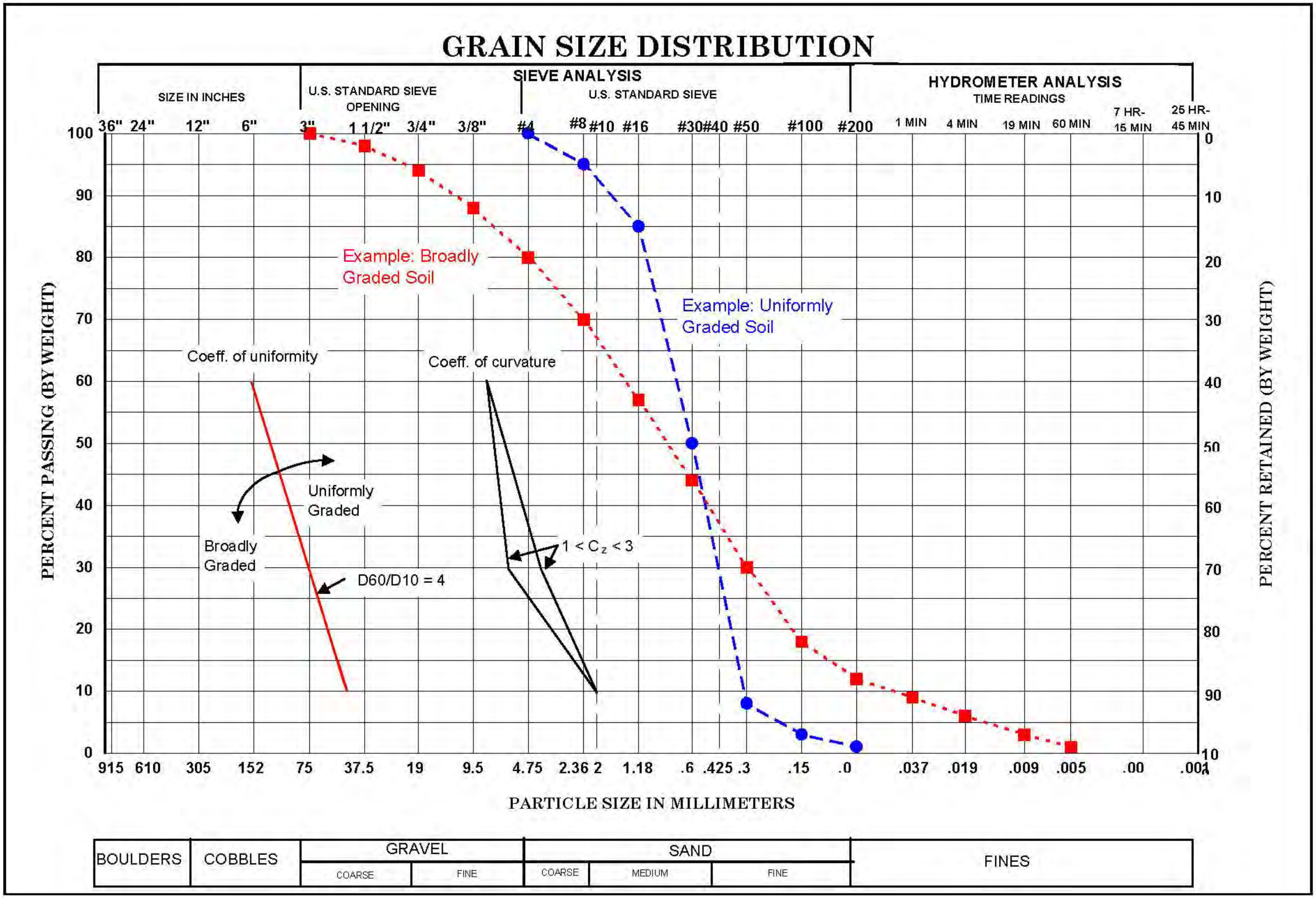

Figure 1. Example of broadly and uniformly graded soils.

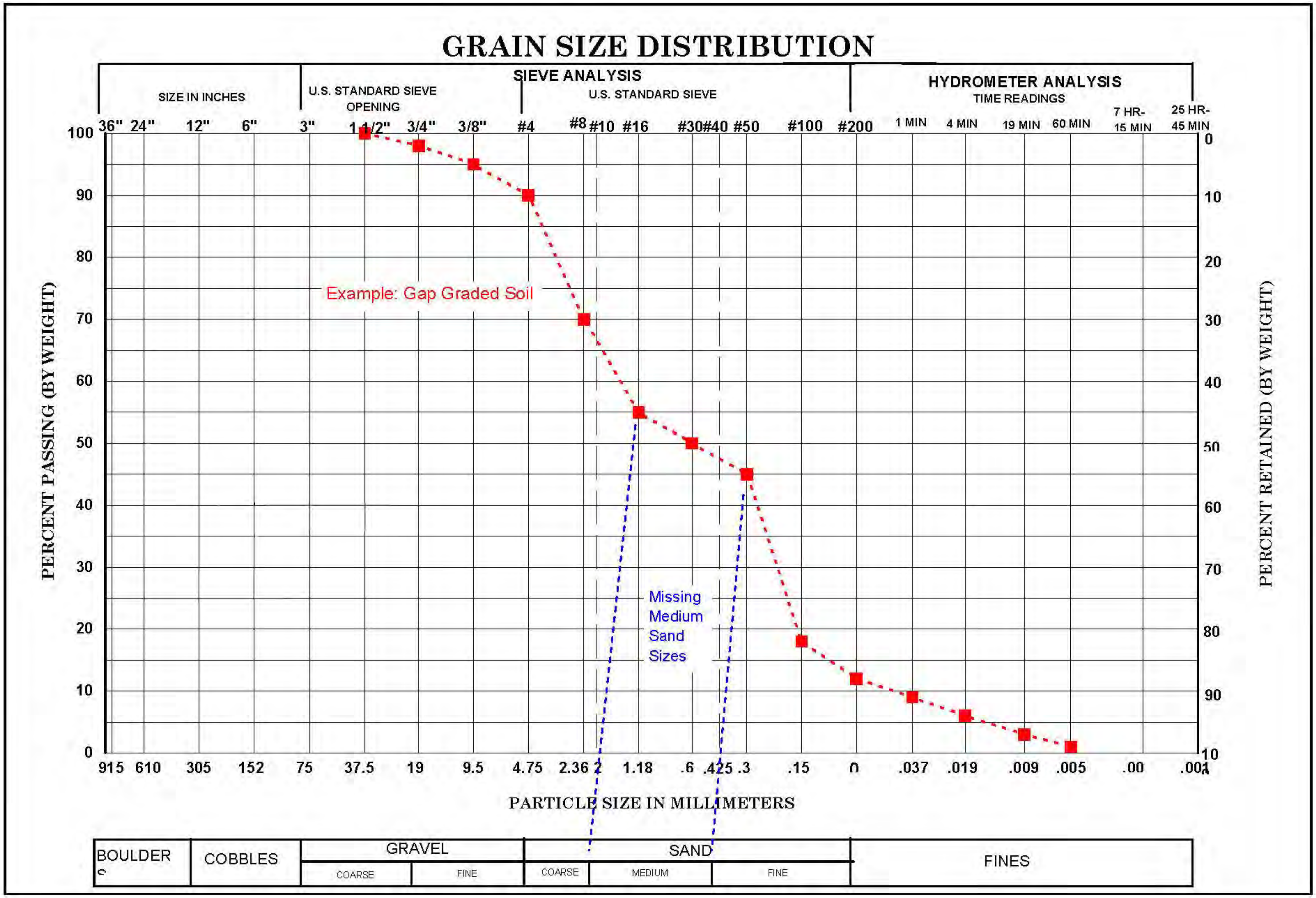

Figure 2. Example of a gap-graded soil where the medium sand sizes are missing.

Gradation symbols

Historical precedence has established a lower case “d” to represent the particle size diameter of the base (soil whose integrity is to be protected) and a capital “D” to represent the particle size diameter of the filter (soil that protects the base). This nomenclature has been repeated by many authors and is commonly used. This nomenclature is satisfactory when designing a single filter for a single base, but may be confusing for two-stage filters since the first stage filter becomes the second stage base. This manual uses the following designation:

$$D_{XX}Y$$

where:

D = Particle diameter
XX = Percent passing for that diameter
Y = Material designation where:
- B = Base
- F = Filter (first stage)
- E = Envelope (or second stage)

Example:

$D_{15}F$ = Particle diameter at 15% passing for a one-stage filter.

Acronyms and Abbreviations

AASHTO	American Association of State Highway Transportation Officials
ASDSO	Association of State Dam Safety Officials
ASCE	American Society of Civil Engineers
CEF	Continuing Erosion Filter
CFRD	concrete face rockfill dam
cm	centimeter(s)
CMP	corrugated metal pipe
c/sec	centimeters per second
CU	coefficient of uniformity
CY	cubic yard(s)
DOT	Department of Transportation
d/s	downstream
EOS	equivalent opening size
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
ft^3/s	cubic ft per second
HDPE	high density polyethylene
HET	Hole Erosion Test

ICOLD	International Committee on Large Dams
kg/cm²	kilogram per square centimeter
mm	millimeter(s)
NEF	No Erosion Filter
NDSRB	National Dam Safety Review Board
NRCS	Natural Resource Conservation Service
pcf	pounds per cubic ft
PI	plasticity index
PSD	particle size distribution
psi	pounds per square in.
PVC	polyvinyl chloride
Q/A	quality assurance
Q/C	quality control
Reclamation	Bureau of Reclamation
SCS	Soil Conservation Service
SEV	sand equivalent value
TAC	top of active conservation
u/s	upstream
USACE	U.S. Army Corps of Engineers
USCS	Unified Soil Classification System

USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
VPI	Virginia Polytechnic Institute

Symbols

k	Hydraulic conductivity (soil permeability to water)
i	Gradient, the ratio of head loss over the distance (length) that head loss occurs: ($\Delta h/\Delta l$)
D_{85}	The particle size diameter in millimeters of the 85th percentile passing grain size
$D_{85}B$	The particle size diameter in millimeters of the 85th percentile passing grain size of the base soil
A	The percentage of soil passing the No. 200 sieve, fines content.
$D_{15}F$	The particle size diameter in millimeters of the 15th percentile passing grain size of the filter
$D_{15}B$	The particle size diameter in millimeters of the 15th percentile passing grain size of the base soil
C_u	Coefficient of uniformity, as determined from a grain size analysis, equal to the ratios D_{60}/D_{10}, where D_{60} and D_{10} are the particle diameters corresponding to 60 and 10% finer on the cumulative gradation curve, respectively
C_c	Standard symbol for coefficient of curvature, replaced in this manual with the symbol C_z to avoid confusion with the compression index symbol C_c
C_z	Coefficient of curvature (also coefficient of gradation), as determined from a grain size analysis, calculated from the relationship: $C_z = D_{30}^2/(D_{60} * D_{10})$

Where D_{60}, D_{30}, and D_{10} are the particle diameters corresponding to 60, 30, and 10% finer on the cumulative gradation curve, respectively.

D_{60}	The particle size diameter in millimeters of the 60th percentile passing grain size
D_{10}	The particle size diameter in millimeters of the 10th percentile passing grain size
$D_{85}E$	The particle size diameter in millimeters of the 85th percentile passing grain size of the envelope (second stage)
$D_{15}E$	The particle size diameter in millimeters of the 15th percentile passing grain size of the envelope (second stage)
G_s	Specific gravity
e	Void ratio (ratio of the volume of soil voids to the volume of soil solids)
γ_b	Buoyant unit weight of soil (saturated soil density minus water density)
γ_w	Density (unit weight) of water

Unit Conversion Factors

Multiply	By	To Obtain
cubic ft	0.02831685	cubic meters
cubic in.	1.6387064 E-05	cubic meters
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
degrees Fahrenheit	(F-32)/1.8	degrees Celsius
feet	0.3048	meters
foot-pounds force	1.355818	joules
gallons (U.S. liquid)	3.785412 E-03	cubic meters
hectares	1.0 E+04	square meters
inches	0.0254	meters
inch-pounds (force)	0.1129848	Newton meters
miles (U.S. statute)	1,609.347	meters
miles per hour	0.44704	meters per second
ounces (U.S. fluid)	2.957353 E-05	cubic meters
pints (U.S. liquid)	4.73176 E-04	cubic meters
pints (U.S. liquid)	0.473176	liters
pounds (force)	4.448222	Newton
pounds (force) per ft	14.59390	Newton per meter
pounds (force) per in.	175.1268	Newton per meter
pounds (force) per square ft	47.88026	Pascal
pounds (force) per square in.	6.894757	kilopascals
pounds (mass)	0.45359237	kilograms
pounds (mass) per cubic ft	16.01846	kilograms per cubic meter
pounds (mass) per cubic in.	2.757990 E+04	kilograms per cubic meter
pounds (mass) per square ft	4.882428	kilograms per square meter
quarts (U.S. liquid)	9.463529 E-04	cubic meters
square ft	0.09290304	square meters
yards	0.9144	meters

1 Purpose and Theory of Filters

1.1 General purpose and function of filters

Filters and drains have been recognized as a means of controlling and directing the flow of seepage water through dams for hundreds of years. Filters are used to prevent movement of soil particles from or between various zones and foundations of embankment dams. Such movement, if not controlled, can result in the development of concentrated leaks that can lead to serious consequences and, in extreme cases, failure of an embankment dam. In fact, approximately 50% of all dam failures are attributed to excess seepage (Fell and Foster 2000). These failures are progressive in nature and begin with the erosion of a few grains of soil, usually undetected. The loss of those soil grains leads to greater seepage, which leads to more soil erosion. This process continues until it is noticed, but usually by this stage, it is too late, and complete failure of the dam cannot be prevented. An embankment dam or other water retention structure that is well constructed from appropriate materials and placed on a sound foundation and abutments may be successful without the use of filters. Many dams that are performing successfully have been constructed without filters. However, it is known that many dams crack, are sometimes poorly constructed, may be constructed from highly erodible material, or may have foundation conditions that allow large amounts of underseepage. These conditions are known to produce the potential for severe distress that can lead to eventual failure of dams. Therefore, design elements such as filters are used as a defensive measure to protect these types of structures from the less than desirable conditions that may exist or develop over the life of the structure. This manual presents discussion of the proper design of embankment dam filters.

The information in this manual applies to granular filters manufactured from natural earth materials by grading, screening, washing, and/or crushing. It covers design principles for meeting particle retention and drainage criteria, quality of materials, the use of filters in dams, and construction considerations. The term *filter* as used in this manual includes a soil gradation that meets both particle retention and drainage criteria. Historically, the terms *filter/drain* and *drain* have been used, sometimes interchangeably. In this manual, the term *drain* refers to a soil gradation that is typically a second stage to the first stage *filter* and is used to convey larger amounts of seepage.

The filter design criteria presented here can be applied to the design of a wide variety of granular filters and drains that are included as elements for many hydraulic structures. While the criteria and procedures in this manual were initially developed for use in embankment dams, they can also be used for drainage elements under spillway slabs, protection of levees against blowout, design of riprap bedding, as well as many other applications.

The design challenge for an embankment dam is to develop a safe cross section that can be constructed from materials available to the site at minimum construction and maintenance costs. One of the most critical requirements of a safe design is the provision of appropriate internal filtering and drainage to control the saturation level and the seepage pressure at a safe level and to prevent the removal of fine soil particles from the critical zones in the embankment and the foundation. Economical design requires the use of materials that protect against failure yet are easily constructed. Since filter materials are some of the costliest materials used in a dam, effort is placed in minimizing the amount of material used. Therefore, the balance of cost, constructability, and reliability go hand-in-hand in providing an economically safe structure.

The main function of filters is to prevent movement of soil particles due to water flow within and beneath embankment dams or other water-retaining structures. Soil particle movement can occur through two basic mechanisms: backward erosion piping and internal erosion. Backward erosion piping occurs when soil particles are detached at the seepage exit or seepage discharge face of intergranular seepage (water seeping through the pores of the soil). Internal erosion occurs when soil particles become mobile due to excessive flow rates. Filters provide protection against these two anomalies progressing toward development of a concentrated (large) leak that could cause excessive loss of water or eventual failure of the structure.

A properly designed filter consists of a granular porous media with pore size openings small enough to prevent migration of the base soil through which water is flowing into the filter. At the same time, a properly designed filter will be sufficiently pervious to offer little resistance to water flow.[1] The design of filters has evolved over time. Filters were first included in the design of water impoundment structures to address the problem of backward erosion piping in foundation soils that were susceptible to this problem. Later, designers recognized a second mechanism of failure described as internal erosion. These two basic mechanisms are summarized in the following sections.

Filters serve to accommodate high gradients through a dam by intercepting the seepage flow from the zone containing high gradients (the changes in hydrostatic head over a given distance) and reducing them to near zero in the drainage system. The water stopping element of the dam is typically a fine-grained soil that is subjected to a high gradient since the pressure head through the dam must be reduced from the reservoir level on the upstream side to the tail water elevation on the downstream side. Placing a filter against the fine-grained soil (core zone) prevents the movement of soil particles and protects it against erosion caused by these high gradients.

Additionally, there is a requirement that filter material be of sufficiently high quality so it will not be able to sustain a crack. In the past, material quality was measured by maximum fines content and plasticity. More recently, it has been found that other types of binders or cementing agents, which were undetected by earlier test procedures, can also result in material that can sustain a crack.

Historically, filter research has focused on the issue of protecting the fine-grained core section of a dam because dam core failures have been experienced multiple times. The filter protection concepts developed from that work are not limited to dams. These same principles can be used in a wide variety of other engineering applications. Filters are used not only to protect the core zones of embankments, but other important zones as well, such as toe and blanket drains. Other sections of the manual discuss the various types of filter zones.

[1] Filters are designed for stability (particle retention) as well as permeability (water flow). Chapter 5 provides detailed considerations.

1.2 Historical research and development of filter design

Early researchers determined that a properly designed layer of material covering an area where seepage is discharging could block the movement of the base soil materials while allowing seepage water to continue to be discharged safely. This layer was termed a filter because it was capable of blocking the movement of the base soil particles. Most of the early filter research investigated material designs that were both sufficiently fine to block the movement of the base soil particles and sufficiently permeable to freely pass the seepage water. These studies focused on determining the grain size of a filter required to protect a base soil. The most commonly studied base soils were relatively low fines content slightly silty, fine, poorly graded sands because those materials were the most susceptible to backward erosion piping.

The concept of particle retention can be envisioned by considering a container of equally sized spheres. The space between the spheres (voids) will have a fixed maximum opening size based on the diameter of the spheres. The size of a smaller sphere that can pass through these voids can then be calculated. While this is a simple mathematical procedure, since soil particles are not spherical or all of one size, the theoretical application to earth materials is limited. Therefore, development of filter criteria for soils centered on empirical relationships based on laboratory testing.

The chronology of these filter studies is summarized in Table 1. Additional information is provided in the links given in the table.

1.3 Seepage and particle movement

Filters are designed to prevent particle movement from intergranular seepage flow where no defects are present in the base soil and seepage water flows only through the pore space of the soil mass. Flow may occur through zones in an embankment or through its foundation. If a soil susceptible to backward erosion is not protected by a filter, the energy of the water moving through the soil may be adequate to dislodge and remove particles at the discharge face. The energy of water moving through the soil is usually expressed by the seepage gradient, which is the available pressure head at a particular location divided by the length of seepage path to that location (usually where it outlets to the atmosphere or into another zone). Along any given seepage flow path, each soil will have a critical

Table 1-1. Chronology of filter studies.

Year	Feature/researcher/organization
	Overview {Link_001}
1902	Bohio Dam {Link_002}
1925	Terzaghi {Link_003}
1934	Harza {Link_004}
1940	Bertram {Link_005}
1941	U.S. Army Corps of Engineers {Link_006}
1955	Bureau of Reclamation {Link_007}
1965	Kassif {Link_008}
1982	Vaughan and Soares {Link_009}
1984	Sherard {Link_010}
1990	Peck {Link_011}
1994	International Committee on Large Dams {Link_012}
2001	Foster {Link_013}
2003	Milligan {Link_014}

gradient based on its properties where, if exceeded at the discharge point, soil particles will be eroded away with the flowing water.

For silts and clays having a plasticity index (PI) greater than about 7, very high gradients are required to initiate backward erosion piping. These gradients are usually not achieved in conventional embankment dams and embankment dam foundations. There are many impoundment structures that were constructed without filters that perform well because the gradients are not large enough to produce piping conditions. Casagrande, in a panel discussion at the Mexico City conference (Casagrande 1969), expressed his views on the potential for intergranular seepage to cause piping in a clay core of an embankment as follows:

> *As a matter of fact, I am not afraid at all of the water that percolates through the clay core if there are no cracks.*

For cohesionless soils (PI < about 7), and particularly non-plastic soils, much lower gradients will initiate backward erosion piping, which can develop into a concentrated leak removed of soil particles and eventual release of stored water. The critical gradient in these soils is dependent on uniformity of particle size, mass and size of particles, and density (to a lesser degree). Soils comprised of particles of fine, uniformly graded sand with no cohesive binder (typically classified as SP or SP-SM in the Unified Soil Classification System) are very susceptible to being detached because of low particle mass and lack of interparticle attraction. Larger sand particles or gravels are more resistant to particle detachment because of their greater mass.

Well-graded sands are more resistant to backward erosion piping because the small particles most susceptible to detachment cannot easily migrate through the soil body to the discharge face because they are blocked by larger particles in the mass. Soils that have been compacted or otherwise are naturally dense usually have more resistance to backward erosion piping.

Accurately defining conditions in which backward erosion piping may be a problem is difficult. The foundation of an impoundment structure may be mostly clay, indicating that a filter would not be needed particularly if the impoundment structure is small. However, undetected silt or sand layers may result in a vulnerable condition.

1.3.1 Protection against backward erosion piping

Granular filter material is placed in contact with a surface of the base soil where seepage water will be percolating through the pores of the soil. During construction, compaction is used to ensure a positive contact between the filter and the base soil (see Chapter 7). A properly designed filter is cleaned of fine particles so that there are insufficient fine soil particles to bind the granular filter particles together and prevent free flow of water.

As seepage flow patterns develop through embankments that impound water and through their abutments and foundations, seepage gradients may become large enough to exceed the critical gradient of the soil at the discharge point. When the discharge face is not supported by a filter and the critical gradient is exceeded, soil particles are eroded by seepage water from the discharge face, forming a cavity or “pipe” that progresses from downstream to upstream at a faster and faster rate as the gradient is increased with the loss of soil. Eventually, a concentrated leak develops in a pipe-shaped cavity that is formed if the soil is capable of supporting such a cavity, and failure usually follows as the cavity enlarges rapidly from the intense erosive forces. This phenomenon is called “piping.” Research (Sherard et al. 1984) has shown that a properly graded filter will restrain the discharge face and preclude the movement of soil particles, preventing piping as seepage water is collected into the drainage system and carried to a safe outlet.

If filters are in contact with the soil subjected to intergranular seepage flow, they support the discharge face with points of contact spaced at some distance determined by the gradation of the filter (particles supporting particles). Apparently, there is some bridging between the contact points where the filter is in contact with the discharge face to prevent any particle movement, including the very small colloidal particles. Coarser filters or other materials that do not support the discharge face with closely spaced contact points will not prevent soil particles from moving when the gradients exceed the critical gradient. Filters used in drainage systems are confined by a downstream zone such that positive pressure is ensured because the cover over the filter is large enough to prevent the seepage pressure from exceeding the confining pressure of the drain and cover. If sufficient pressure is not provided, the filter will not support the discharge face of the base soil, and protection against backward erosion piping will be lost.

Figure 1-1 illustrates how the filter in contact with the soil discharge face provides support and prevents soil movement.

1.4 Preferential flow and internal erosion

Filters are also designed to prevent particle movement from preferential flow and internal erosion along cracks, anomalies, or defects in the embankment. Preferential flow paths can occur in earth embankments, their foundations, or at contacts between the fill and concrete structures or bedrock. In this mechanism of soil erosion, soil particles are detached by slaking along the preferential flow path (i.e., along the walls of a crack in the soil), and the soil is subsequently eroded by water flowing at relatively high velocity (compared to the velocity of flow in intergranular flow). The eroded particles are then carried through the preferential flow path to the filter face. Most soils are subject to erosion from this mechanism, and modern filter criteria were developed to protect against this type of erosion. Figures 1-2, 1-3, and 1-4 illustrate the way in which a filter works to prevent internal erosion (Sherard et al. 1984a).

Some early studies were unsuccessful in defining filter boundaries for silts and clays because, for intergranular flow (without a defect in the base soil),

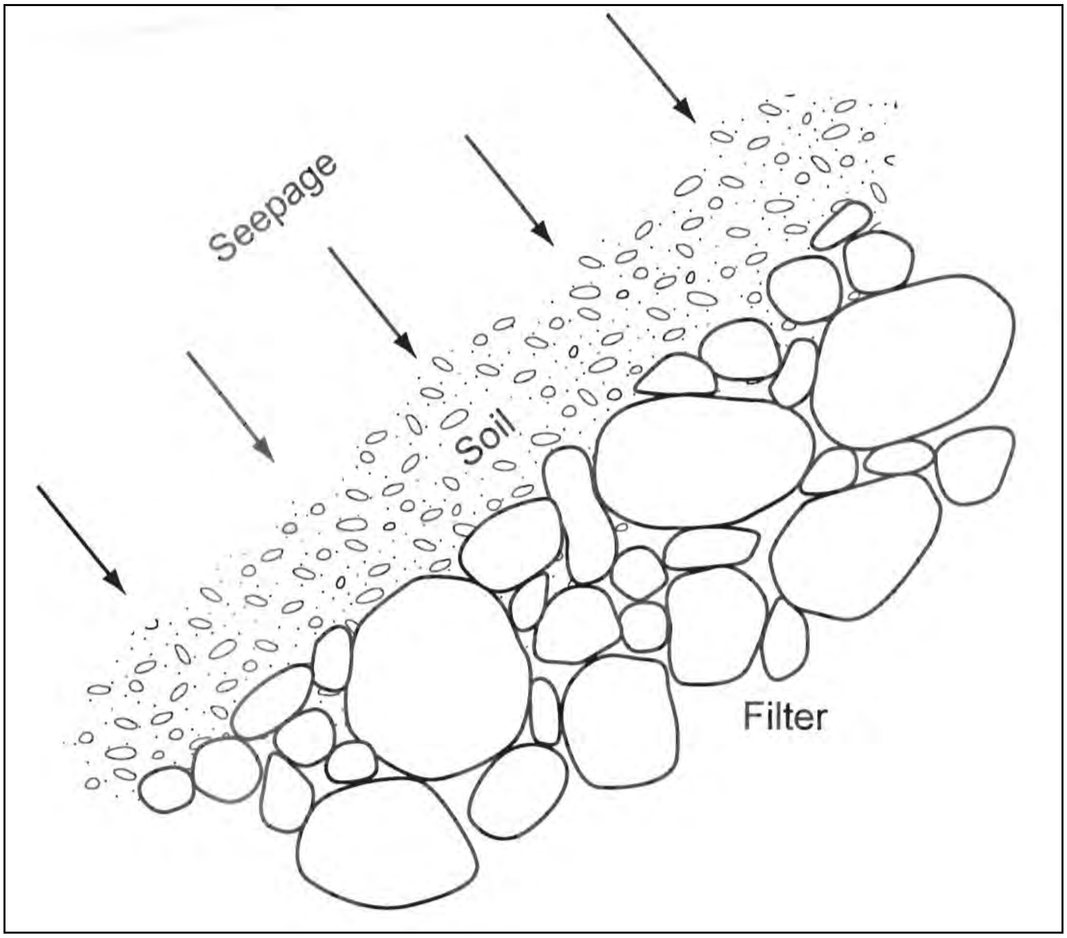

Figure 1-1. Schematic demonstrating the manner in which a properly designed filter prevents the movement of base soils by seepage forces at the discharge face. The filter supports the discharge face with closely spaced contact points as compaction melds the two zones together such that bridging between the contact points prevents any movement of base soil particles into the filter. At the same time, the filter is sufficiently coarse to allow seepage water to escape freely.

failures could not be induced in laboratory specimens even when used with a very coarse filter. The Waterways Experiment Station Study reported in TM 183-1 (U.S. Army Corps of Engineers 1941) noted that attempts to define filter failure boundaries for a loess (silt) and a sandy loam were unsuccessful because the low permeability of the soil meant that backward erosion piping could not be induced under the low gradients being used.

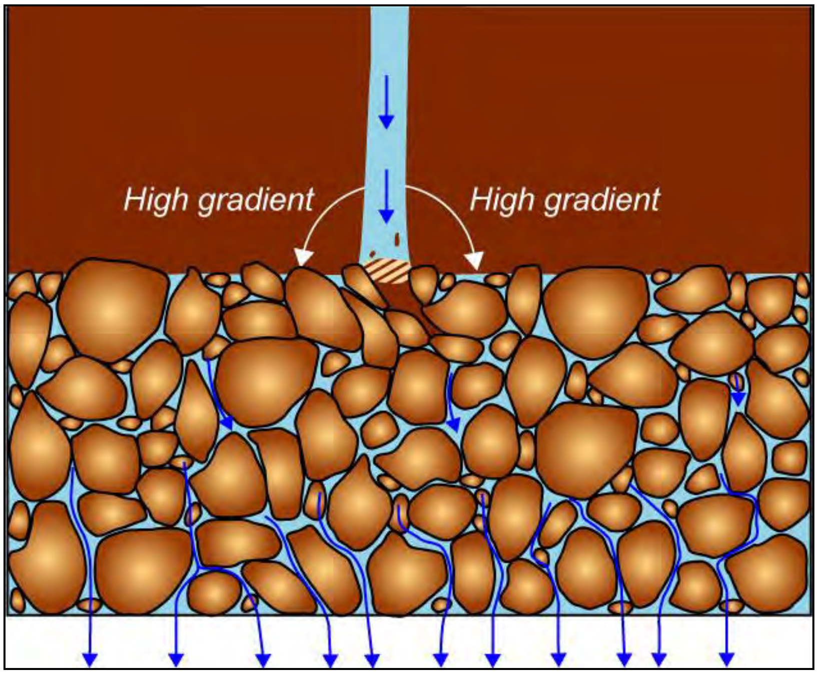

Figure 1-2. Eroding soil in the crack is caught at the filter face, stopping flow in the crack. High gradients cause hydraulic fracturing from the crack to the adjacent filter.

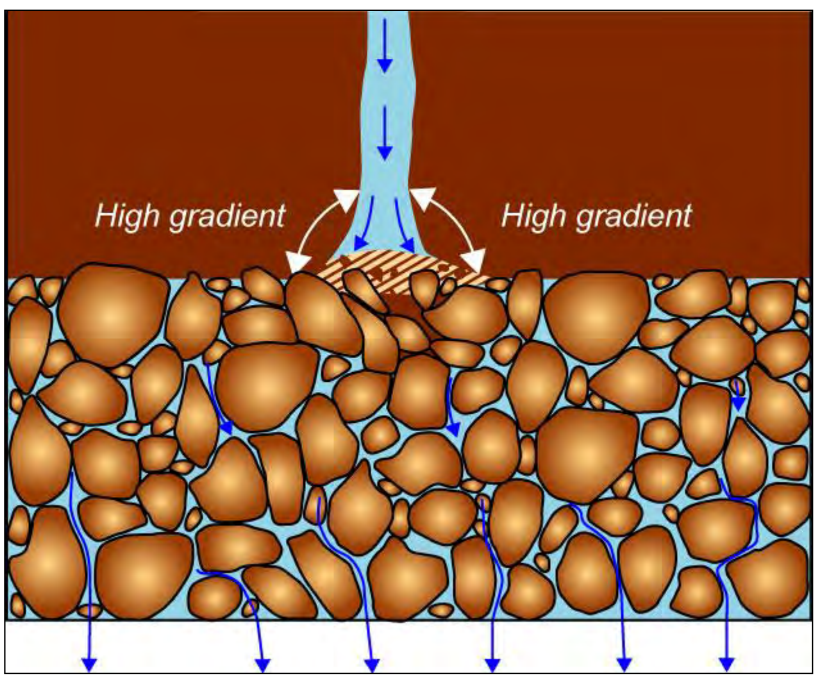

Figure 1-3. Eroding soil from a crack has been caught at the filter face, and hydraulic fracturing from high gradients between water in the crack and the adjacent filter has caused some widening of the filter cake near the crack.

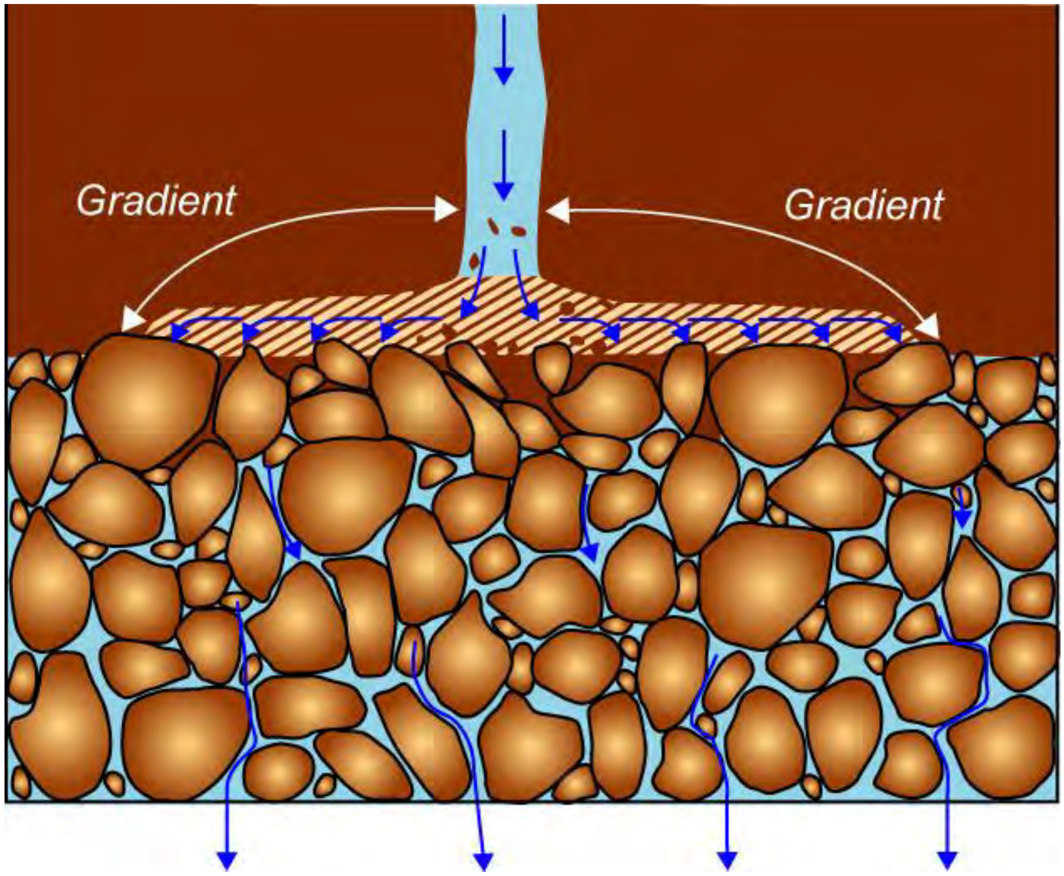

Figure 1-4. Eroding soil from the crack has been caught at the filter face, and hydraulic fracturing from the high gradients between water in the crack and the adjacent filter has caused further widening of the filter cake until the gradient is reduced. The filter cake having a very low permeability covers the width of the crack and some distance on each side of the crack. The remaining filter is open for collecting seepage flow through the pores of the soil between cracks.

In Sherard's June 1984 article, "Filters for Silts and Clays," (Sherard 1984b), the observation was made:

> *As a first effort, a number of conventional filter tests (without an initial hole in the base specimen) were made with compacted sand and sandy gravel filters using relatively thin (30-60-mm thick) base specimens of clay and silt. The specimens were compacted near Standard Proctor Optimum water content. In these tests, the water pressure acting across the base specimen was gradually increased to a maximum of about 6 kg/cm, giving a hydraulic gradient of about 1,000-2,000.*

> *At relatively low pressures, generally below 1.0 kg/cm, no filter failures occurred, even for very coarse filter tests lasting many weeks. The small quantity of water seeping from the base sample into the filter had very little energy, and there was no tendency for the fine clay or silt base material to enter the filter pores.*

In this same article, research showed that if water pressures were increased to a point in which the base soil was hydraulically fractured, concentrated flow would occur in the cracks that would erode through filters that were overly coarse. Tests used for studying the compatibility of filters and base soils are described in Chapter 4.

1.4.1 Protection against cracking and internal erosion

Embankment dams should be protected against erosion and cracking. In the Seventh International Conference on Soil Mechanics and Foundation Engineering held in Mexico City in 1969, Casagrande stated:

> *It is not possible to prevent entirely the formation of substantial tension zones and transverse cracks in the top of the dams in the vicinity of the abutments, no matter what materials we use in the dam. Therefore, we must defend ourselves against the effects of cracks.*

Cracks or other preferential flow paths are more likely at the following locations:

- Upper part of the embankment
- Overly steep abutments or above abrupt changes in the foundation or abutment profile
- At the embankment/abutment contact
- At the embankment/foundation contact
- Around and above a conduit or other structural penetration through the embankment
- At the contact between the embankment and spillway or abutment wall
- Narrow and/or steep cutoff trenches

During construction and during the first few years of service, particularly the first filling of the reservoir, settlement is occurring in the dam and foundation.[1] Differential settlement can occur over short distances due to differing settlement characteristics of foundation soils or abutments with variable or steep slopes. These movements in the dam cause stress release. The stress release may be both in the horizontal as well as the vertical direction. Vertical stress release is caused by arching between two or more locations that do not settle as much as a location between them. An outlet works conduit is usually a vulnerable location for stress release and cracking. Since the conduit passes all the way through the dam in a transverse direction, it is a particularly critical area for cracking and concentrated leak development. Sherard (1986) provides a thorough discussion of this phenomenon. In addition to transverse cracks, longitudinal cracks can also develop due to differential settlement or slope instability. Longitudinal cracking is typically not as serious as transverse cracking due to common seepage paths through dams.

Most hydraulic fracturing occurs during the first filling of the reservoir as a wetting front passes through the dam. As water under pressure encounters unsaturated soil of the dam, hydraulic fracturing occurs when the water pressure exceeds the soil pressure. Many existing flood control dams have not filled and thus these structures may have a higher risk of failure.

Internal erosion may initiate in zones of poor compaction or coarse lifts. Other zones of poor compaction can occur in exposed surfaces during winter shutdown, diversion gaps, and transverse joints. Openings may result from overhangs on rock abutments or along structures or penetrations through the dam around which the earthfill is poorly compacted. The zone under the haunches of pipes that do not have structural cradles or concrete encasement is a common location for voids and poor compaction. Animal burrows and root holes are also possible causes of openings in embankments.

Some cracks may be very narrow, particularly those caused by hydraulic fracturing. Water penetrating the sides of the crack may initiate some swelling of the unsaturated soil that could close the crack before erosion begins to make it wider. The closing of cracks in this manner has likely

---

[1] Note that flood control dams may not fill until many years after they are constructed. Since they have not received this critical first filling, they should be considered “new” until that time.

saved many dams over the years, but cannot be depended upon with any certainty because it is a race to see which process progresses faster, swelling or erosion. For dispersive soils, the erosion will generally always win, which has resulted in the failure of many dams constructed of such materials. For more plastic soils, the reverse is usually true.

Desiccation cracking can occur in the crest of dams constructed of higher plasticity clay in arid environments. These types of cracks can develop over extended periods of time, will usually be worse in extended dry periods, and typically occur in the upper part of the embankment above the normal water surface. For these reasons, problems can occur during flood events that raise the reservoir to elevations not seen historically. Water can then flow through the desiccation cracks, leading to failure of the dam without floodflows overtopping the dam.

The problems introduced by cracking in embankment dams are remedied with the use of a filter. Since proper filter application will stop particle erosion through a crack, a process known as self-healing, failure of the dam is prevented. Therefore, embankment dams should include filter materials to protect all zones subject to cracking for any reason. The filter must be constructed of free-flowing sand with low fines content that will not bind together if a crack occurs. The filter materials should be installed at locations where it will protect all vulnerable areas. This issue is addressed more thoroughly in Chapter 2.

1.5 Seepage collection and pressure reduction

Another main function of filter protection in dams and impoundment structures is to provide for the collection of seepage water in such a way as to reduce the seepage pressure in the downstream section of the dam and carry the water to a safe and controlled outlet. In order to do this, the filter and drainage system must have a permeability larger than any of the layers in the dam or foundation that encounters the filter. When the filter zone next to the soil has a permeability lower than some of the base soil strata, pressure will build up in those layers with higher permeability. This unsafe condition may also exist if the filter drainage system does not have sufficient capacity to carry the volume of seepage water.

2 Types of Filters and Applications

2.1 Introduction

This chapter will address the issues related to the use of filters and the different types of filters used in dams. It is recognized that the cost of filter material, and how that contributes to the overall project cost, is an important issue, especially for smaller dams. For these dams, especially in remote areas, the cost of filter materials can be a significant portion of the total project cost. In the interest of reducing costs, the designer can feel pressured to reduce or even eliminate the use of filter material. While cost is an important issue, the need to provide a safe structure is more important.

Historically, many small dams (<50 ft high) have been built without any filter or drainage zones, especially those constructed prior to 1980. Additionally, many mid-size dams (50 to 300 ft high) have been built without "modern" filters, although they do contain graded transition zones. Many of the dams in each of these categories have performed successfully for many decades. On the other hand, there have been notable dam failures, including all dam sizes that have resulted in loss of life and extensive property damage. The failure of dams built without filters led to the general design practice for embankments to change in the 1980s. While mid-size and large dams, which are almost always high-hazard structures, are now constructed with extensive filter elements, some question the level of protection required for small dams, primarily due to the cost issue. It should be noted, however, that since the advent of the dam safety movement in the late 1970s, the failure rate of embankment dams due to piping has remained about the same. The reason for this can be two fold. First, as dams age, they deteriorate due to undetected internal erosion and over time eventually fail. Second, smaller structures continue to be built without adequate filter protection and fail upon first filling.

Additional discussion of the function of filters is presented here {Link_025}, filter use in terms of dual function is presented here {Link_026}, and the design to satisfy function is presented here {Link_027}.

2.2 Filter and drainage zones

In the past, the use of filter protection in embankment dams has been decided on whether or not the facility is either low or high hazard. A concern with this philosophy is how the hazard classification can change with time. As rural areas grow and urban areas spread, many low-hazard dams are re-classified to high-hazard dams. The dam owner is then faced with the challenge of upgrading a deficient structure, usually at a significant cost. Therefore, it is recommended that all new embankment dams, regardless of size or hazard classification, be designed with protective filters.

Often during safety evaluation of existing dams, questions arise about whether filters should be added. Due to the satisfactory performance of many dams that do not include filters, typically an identified deficiency must be present in these dams to justify the addition of filters. Dams with conduit deficiencies would have a protective filter diaphragm added. Seepage deficiencies through the foundation should be addressed with the addition of a toe drain, and for embankment seepage deficiencies, a chimney should be used. Additionally, for older dams in metropolitan areas with a large downstream population, and attendant consequences, filter protection is added even when no known deficiency has been identified.

Reclamation (2007a) lists filter classes as follows:

> *Drainage filters (class I) – Filters whose purpose is to intercept and carry away the main seepage within a dam and its foundation. These filters may have to remove large amounts of seepage for dams on pervious foundations or dams of poor construction. The filters consist of uniformly graded materials, typically in two stages. The filter must meet the requirements for both particle movement and drainage. Toe drains typically fall into this class.*
>
> *Protective filters (class II) – Filters whose purpose is to protect base material from eroding into other embankment zones and to provide some drainage function in order to control pore pressure in the dam. These filters are typically uniformly graded and in several stages, but they can also be broadly graded in the interest of reducing the number of*

zones to make the transition to the base material. This class includes chimneys, blankets, and transition zones on the downstream side of a dam.

Choke (inverted) filters (class III) – Filters whose purpose is to support overlying fill (the base material) from moving into pervious or open work foundations. These filters are typically broadly graded and have a requirement only to stop particle movement. There is no permeability requirement. Choke filter material is also used in emergency situations in an effort to plug whirlpools and sinkholes.

Seismic crack stoppers (class IV) – Filters whose purpose is to protect against cracks that may occur in the embankment core, especially caused by seismic loading and/or large deformations. The dimensions of this class of filter are controlled by expected displacement (horizontal or vertical). While there is no permeability requirement for this type of filter, it should be relatively free of fines so the zone itself does not sustain a crack. A second stage (gravel) filter may be required if concern exists that the first stage finer zone might sustain or allow propagation of a crack. Second stage filters may also be required for transition to a coarser shell material. This class of filter is typically used for chimneys and transition zones.

The following two sections describe, in general, filter protection as it is used for new and existing dams. A specific description of embankment elements is presented in Section 2.3

2.2.1 New dams

For new projects on sites having the following **undesirable situations**, filters will be **necessary**:

- The core zone of the embankment is non-plastic (plasticity index [PI] < 7). Soils are not available to construct a core zone in the dam and a rolled fill cutoff trench with higher PI values.
- Embankment and/or foundation soils are dispersive clays.

- Foundation soils are erodible and/or susceptible to piping, and an effective cutoff of seepage is not present.
- Potential for differential settlement in a transverse direction to the embankment. Conditions that can lead to differential settlement include steep bedrock profiles, problematic foundation horizons such as soft clays, or collapsible soils. Differential settlement ratios greater than 1.0 ft per 100 ft are excessive.
- Hydraulic fracture of the core zone is likely based on the potential for arching of zones in the embankment.
- Artesian pressures under or downstream of the dam beneath structures or clay horizons.
- Any penetration through the embankment, including conduits used as either outlet works or spillways.
- Pervious (sand, gravel, and/or cobble foundation layers) foundations.
- Highly jointed or fractured bedrock foundations, including those types of foundations that have been grouted.
- Dams in areas of significant earthquake loading (> 0.25 g) that provide sufficient energy that could lead to cracking of the embankment.
- Dams located on active faults.
- Dams on rock foundations where the geologic processes over time have resulted in tensile zones near the rock surface (pull apart).
- Dams on soil foundations subject to liquefaction.

Table 2-1 summarizes conditions and types of filter used to protect against these conditions. Note that the listed conditions are independent of one another and, if multiple conditions are present at a site, then combinations of filter types will be required.

2.2.2 Existing dams

There are slight differences for application to new construction and modification to existing dams. For new construction, the chimney would be placed near the centerline of the dam for central core designs, whereas the addition of a chimney to an existing dam would require removal of a large portion of the existing embankment to obtain this location. The central location is desirable to maximize the confining stress on the chimney as well as to minimize hydrostatic pressure in the downstream shell. Therefore, modifications to existing dams will typically locate the chimney further downstream than what would be used for new construction. When chimneys are located downstream, sufficient overburden must be provided

Table 2-1. Conditions encountered in embankment dam zones and how they are protected by filters.

Feature	Condition	Possible Consequences	Type of Filter Needed
Embankment	Impervious core composed of nonplastic (PI ≤ 10) materials	Particle erosion, cracking	Chimney, blanket, toe drain
Embankment and/or foundation	Composed of dispersive clays	Particle erosion	Chimney, blanket, toe drain
Foundation without cutoff	Composed of erodible materials	Particle erosion	Blanket, toe drain
Embankment and/or foundation	Potential for differential settlement of impervious core[a]	Vertical cracking in impervious core	Chimney, blanket, toe drain
Embankment	Hydraulic fracturing of impervious core[b]	Horizontal cracking in impervious core	Chimney, blanket, toe drain
Foundation	Artesian pressure	Particle erosion, blowout of toe	Blanket, toe drain
Embankment	Structural penetration by conduit	Cracking, particle erosion	Conduit diaphragm
Foundation	Pervious materials	Particle erosion	Blanket, toe drain
Foundation	Highly jointed/fractured rock	Particle erosion	Blanket, toe drain
Embankment and/or foundation	Seismic loading and/or locations on active faults	Cracking	Chimney, blanket, toe drain
Foundation	Tensile zones near the bedrock surface	Cracking	Chimney, blanket, toe drain
Embankment	Founded on pervious foundation materials	Particle erosion	Choke (inverted filter)

[a] Conditions that can cause differential settlement include steep and/or irregular abutment profiles and problematic foundation conditions such as discontinuous strata and strata composed of materials of varying thicknesses and composition. Generally, differential settlement ratios of 1 ft per 100 ft are considered problematic.

[b] Usually due to arching of impervious core between adjacent zones that are composed of different moduli (normally stiffer than the core).

for stability against a potential full reservoir head. In a similar manner, the blanket added during an existing dam modification would be shorter since the chimney it connects to is further downstream. Examples of the two arrangements are shown in Figures 2-2 and 2-3.

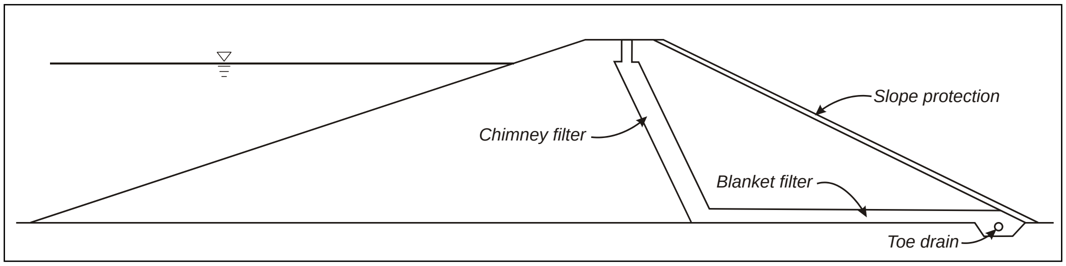

Figure 2-1. Simple cross section showing a chimney used in a new dam.

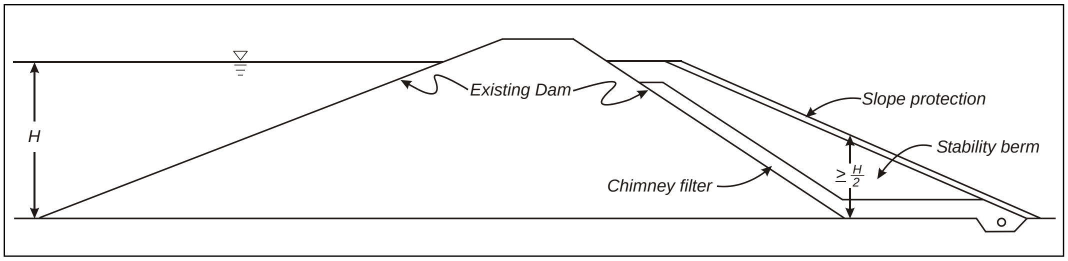

Figure 2-2. Simple cross section showing a chimney added to an existing dam.

2.3 Embankment filter and drainage zones

Embankment dam seepage may be controlled by the use of seepage barriers and filter/drainage zones. Seepage barriers are intended to prevent or decrease seepage, while filter and drainage zones are intended to safely control seepage. The most commonly used categories of filter and drainage zones used in design of embankments are described in this section. Readers are referred to the major U.S. Government embankment design agencies, including the U.S. Army Corps of Engineers (USACE), Bureau of Reclamation (Reclamation), and the Natural Resource Conservation Service (NRCS) (formerly the Soil Conservation Service) for additional information on detailed design methodology. Some designs will include only one component or category of filter and drainage zone, but most designs will include several. The information contained in this chapter is not intended to serve as an embankment design procedure, but rather presents information on how filter and drainage elements fit into the overall embankment design {Link_029}.

Sand or sand and gravel filter and drainage zones are important design elements for many new designs and for repairs or upgrades of existing embankment dams. Figure 2-3 is a composite diagram showing most of the major categories of seepage control zones normally found in central core embankment designs. Rarely would all of these zones be included in any one design. The purpose of Figure 2-3 is to provide a diagrammatic description of the various zones.

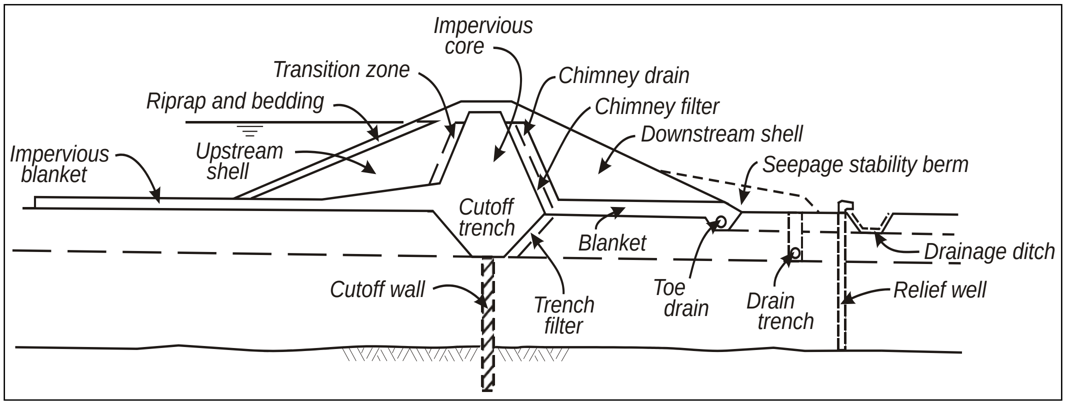

Figure 2-3. Typical embankment dam design elements found in a central core design.

Components of a modern embankment dam illustrated in are:

Core – Zone of low permeability soil that acts as the water barrier in the dam.

Cutoff Trench – A cutoff trench to rock or other low permeability strata that is integrated with the overlying core.

Upstream Shell – Zone of higher strength soil to support the upstream face of the core. The geometry of the upstream core is sometimes dependent on the rapid drawdown loading case.

Transition Zone – A zone on the interior side of the upstream or downstream shells. Upstream transition zones can also function as seismic crack stoppers.

Chimney Drain – Zone that carries away seepage coming through the chimney filter and delivers it to the blanket drain. It also acts as a transition zone between the chimney filter and the downstream shell. Usually, this zone is composed of gravel-size particles.

Chimney Filter – Zone that protects the core from internal erosion and piping. Usually, this zone is composed of sand-size particles.

Riprap and Bedding – Riprap is the zone that protects the upstream slope of the dam against erosion caused by reservoir wave action. Bedding under riprap protects against particle movement of the protected zone after reservoir drawdown.

Downstream Shell – Zone that supports the chimney and downstream slope of the core.

Blanket Drain – Zone that provides foundation hydrostatic pressure relief for pervious foundations and protects against particle movement in soil foundations. It also provides an outlet for seepage water collected by the chimney.

Toe Drain – Collects water from the blanket drain as well as any foundation seepage and safely conveys it away from the embankment.

Drainage Ditch – Open trench downstream of the dam that collects seepage water. It is most effective when it extends into a pervious layer. It may also be used to collect water from relief wells.

Relief Well – Collects seepage water in the foundation that cannot be collected by toe drains due to overlying impervious layers. It is typically used to reduce artesian foundation pressures in confined layers.

Impervious Blanket – Extends the seepage path and increases the head loss zone for dams on pervious foundations when a cutoff under the dam is not practical. Upstream blankets are integrated into the core of the dam.

Cutoff Wall – Vertical water barrier in rock, also known as a grout curtain. The grout curtain will fill all fractures, joints, and other openings in the rock to prevent seepage flow. Cutoff walls are used as the cutoff through soil foundations. Cutoff walls are usually deep trenches backfilled with a soil-cement-bentonite slurry.

Another type of zone often used in modern dam designs is a filter diaphragm around a conduit extending through an embankment. This category of zone is described later in this chapter.

Elements that are needed in a particular embankment design depend on geology, site conditions, available materials for construction, loading conditions, and economics. Detailed embankment design is beyond the scope of this manual. This chapter will address the chimney, blanket, diaphragm, and toe drain elements.

Many embankment designs for seepage control include both foundation and embankment filter/drainage zones that work together to provide a complete system. In addition to filter and drainage zones, most designs employ various methods to intercept seepage and control the quantity of flow and hydraulic gradient. By including both seepage reduction measures and filter/drainage zones, a double line of defense is provided that increases the safety of a structure. A summary of available U.S. Federal Agency design guidance is given in {Link_028}.

2.3.1 Central core dam

Any given embankment zone will have one or more of four purposes: strength, compressibility, water barrier, or drainage. For central core dams, the primary water barrier (also called the core) will have low permeability but, as is typical for such materials, will have relatively low strength. Availability of suitable core material may be limited depending on the site. For these reasons, it is desirable to limit the size of the zone. If abundant material is available, the entire dam can be made out of this single zone, which is known as a homogenous dam. When the size of the core is minimized, the side slopes are steep and require support. Support is provided by upstream and downstream shells. Since the purpose of the shell is to support the core, it only has to provide strength for that purpose. This central core and shell arrangement is illustrated in Figure 2-3.

It is generally desirable to obtain full seepage piezometric head drop near or just downstream of the dam centerline. Depending on the material used to construct the core, this head loss may be achievable by the core itself. If the core material does not fulfill this role, and to provide assurance that the head loss is achieved, drainage zones (also known as chimney drains) are provided on the downstream face of the core. The zone immediately against the core face is termed the chimney filter and provides drainage and particle retention as described elsewhere. If needed, a second zone downstream of the chimney filter is included, known as a chimney drain. These two zones allow maximum piezometric head loss, and are included in the cross section between the core and downstream shell, as illustrated in Figure 2-3.

2.3.2 Diaphragm dam

While the layout of zones previously discussed is different for diaphragm-type dams, the concepts are the same. A diaphragm dam achieves its full head loss near the upstream face of the dam as opposed to the centerline location for the central core layout described previously.

Today, diaphragm dam designs are typically concrete face rockfill dams (CFRD). As the name implies, the diaphragm is a concrete slab on the upstream face of the dam. While the concrete acts as the water barrier, a secondary "impervious" material is used under the slab to attenuate any seepage that may come through the slab joints. Beneath this impervious layer are first and second stage filters that also act as a transition zone to the rockfill section that constitutes the body of the dam.

In the past, some dams have been constructed with the core located in the upstream one-third of the cross section, and in some cases, the core is quite thin, approaching a true diaphragm appearance. This layout is seldom used today in the U.S. due to concerns about upstream slope stability and the high gradients imposed on thin sections. If such a section is used, it must be protected by filters in a manner similar to that used for CFRD.

2.3.3 Embankment chimney filter and drain

Chimney filters are an effective method of protecting an impervious core from potential internal erosion failures and, at the same time, effectively controlling the phreatic surface through the embankment. A typical chimney filter under construction is shown in Figure 2-4. The use of a chimney drain is dependent on the expected amount of seepage through the core; cracking potential, especially related to seismic loading; and composition of the downstream shell. If the downstream shell is coarser than the filter (as defined by the filter criteria in this manual), a chimney drain will be required. If rockfill is used for shell material (due to its high strength and low cost), an additional zone or zones may be required between the chimney drain and the shell. Since the drainage function has been met by the chimney drain, these zones are usually called transition zones. Particle retention criteria should be met between these transition zone(s) and the shell.

Figure 2-4. Multiple zone chimney filter being constructed in zoned dam by concurrent method of construction.

Vertical and inclined geometries are commonly used for design of filter and chimney drains in an embankment dam. Note that while a vertical geometry is similar to a structure chimney, inclined geometries are also called chimneys. The type of geometry used is a function of the dam size, construction method, and core geometry, as described in the next sections. Chapter 7 includes additional discussion of construction considerations for these two geometries.

Vertical chimneys are used most often where impervious core material is scarce and the downstream slope of the core is vertical. Additionally, vertical chimneys are sometimes utilized where the dam is a homogenous impervious structure where the chimney is constructed by the trenching method. The primary advantage of a vertical chimney is that maintaining proper location during construction is more straightforward and dependable than when constructing an inclined chimney. This results in being able to specify a smaller width (say 4 or 5 ft), which requires less material. Disadvantages of a vertical chimney include (1) a geometry more conducive to longitudinal cracking at the impervious core-chimney boundary and (2) a longer horizontal blanket drain is required, which results in a greater quantity of blanket material.

Inclined chimney filter and/or drains have the advantage of lessening the susceptibility of cracking at the impervious core-chimney interface and requiring a shorter horizontal blanket drain that results in a smaller quantity of blanket material. The main disadvantage of inclined chimneys is that more difficulty is involved in maintaining proper location during construction since the chimney location must be moved laterally after placement of every lift. Additional information is available here {Link_029}.

Inclined chimney filter/drainage zones

Inclined chimneys can be constructed in one of two ways along with the adjacent core material and downstream shell. The first is to construct one lift ahead of the adjacent zones and the second is one lift behind, as described in Chapter 7. An inclined configuration for an embankment chimney filter has the following advantages and disadvantages:

- Advantages:
  - Most inclined drains are constructed using the concurrent fill placement method (discussed in Chapter 7). This method involves fewer steps in construction and should be less expensive on a unit basis than a vertical configuration.
  - An inclined configuration lends itself better to constructing wide drains, which can reduce construction-related uncertainty. See Section 6.3 for further discussion on minimum zone thickness.
  - Constructing a two-stage filter is easier.

  - The amount of filter required for an outlet blanket drain or strip drains will be smaller for this configuration compared to a vertical chimney configuration.
  - The contacts (joints) between the chimney filter and the upstream core and downstream zone are in compression and there is less potential for cracking resulting from differential settlement.
- Disadvantages:
  - Staking and maintaining limits of the drain zone during construction are more difficult than for a vertical drain.
  - Contamination of the drain from adjacent embankment zones is a problem because the concurrent method of fill placement is often used.
  - Contamination of the filter from construction crossings is more likely because the drain is always exposed if the concurrent method of placement is used.
  - Care must be exercised to prevent damage to the filter zone from overflow during construction. See the case history on Tallaseehatchie Creek Site 1 in Alabama (Attachment E).

Vertical chimney filter/drainage zones

Vertical chimneys are constructed through core material by placing several lifts and then trenching back through those lifts. The trench is then backfilled with filter material and compacted. This method is also sometimes referred to as the trench back method. This process is repeated until the full height of the chimney is achieved. (See Chapter 7 for additional explanation of this construction procedure.) Note that the trenching will require that the top of the chimney from the previous trench be exposed by the current trench. A vertical configuration for an embankment chimney filter has the following advantages and disadvantages:

- Advantages:
  - Staking and maintaining accurate control of the limits of the drain zone during construction are simpler than for an inclined drain.
  - This configuration lends itself to the cut and fill method of construction, which results in less contamination from adjacent zones of the fill than the concurrent method of construction.
  - A vertical configuration lends itself better to constructing a relatively narrow drain. Using a 3-ft-wide backhoe, the chimney filter may be constructed to a 3-ft or greater width, resulting in a lower

yardage of filter material than a inclined drain. See Section 6.3 for further discussion on minimum zone thickness. Due to a smaller volume of material relative to the inclined chimney, vertical chimneys are less expensive.

- Disadvantages:
  - Differential strain between the filter zone and adjacent embankment zones is more likely to result in surface cracking of the overlying fill than for a inclined drain. Differential settlement can result in crack propagations to the top of the dam.
  - Vertical wall caving during construction can result in an ineffective filter.
  - Potential for lower-permeability soil layers left in trench bottom after each trenching cycle.

2.3.4 Appurtenant structures

Conduit filter diaphragm

Protection of conduits and other penetrations through embankment dams cannot be overstated. These conduits will establish a preferred seepage path directly through the embankment from the reservoir to the downstream toe. This condition was recognized in the past, and the remedy at the time was to include antiseepage collars around the conduit, the idea being that the flow path at the embankment conduit interface would be interrupted. It is now known that the inclusion of these collars prevented compaction equipment from getting next to the conduit, and adequate compaction was not achieved. This results in a low-density zone surrounding the conduit to the outside limits of the collars. A preferential seepage path then exists at the outside limits of the collars. An additional problem results from differential settlement and cracking between the two zones of differing densities. The outcome of this condition is shown in Figure 2-5. While the use of seepage collars has not been permitted since the 1980s, their use continues today. The proper method of protecting a dam against piping failure along conduits is through the use of filter diaphragms.

A filter diaphragm is basically a type of chimney filter in the embankment that is limited in extent both vertically and horizontally, although it should be integrated into a drainage blanket, which will act as an outlet. Filter diaphragms are used on smaller dams and in situations in which filter protection needs to be added to existing structures, as described in the next

Figure 2-5. Embankment dam breached after piping along the conduit. The view is upstream. Note pre-cast concrete pipe placed on a concrete cradle and the use of seepage collars (Photo credit NRCS).

section. It should be noted that when a chimney is used in an embankment cross section, it will surround any conduits and a specific filter diaphragm is not needed. The filter diaphragm surrounds a conduit passing through the embankment, and its purpose is to intercept seepage along the embankment/conduit interface and prevent piping of those soils, as well as intercepting cracks in the surrounding earthfill that could be caused by differential settlement of the embankment caused by the presence of the conduit. A detailed description of conduit diaphragms is included in the companion FEMA Manual, Technical Manual: Conduits Through Embankment Dams, FEMA 484 (2005). For the addition of filter protection to an existing unprotected conduit, see Section 3.2.

Filter considerations near concrete dam sections

Special attention must be given to the junction of embankments with concrete structures such as outlet works, spillway walls, lock walls, and powerhouses to avoid piping along the zone. An embankment abutting a high concrete wall creates a tension zone in the top of the embankment similar to that occurring next to steep abutments. Horizontally battered concrete contact surfaces will ensure that the fill will be compressed against the wall as consolidation takes place. The interface of an earth embankment and a concrete structure should be aligned at such an angle that the water load will force the embankment against the structure to reduce seepage along this interface. An embankment wraparound to transition from a concrete dam to an adjacent earth embankment is recommended, as shown in Figure 2-6. A filters or drain provided downstream of the embankment core and beneath the downstream portion of the embankment should be carried around to the downstream contact with the concrete structure.

Other structures

Filter and drainage zones are frequently placed around appurtenances to provide protection along the structure. Such structures include spillway chutes and outlet works stilling basins. Figure 2-7 shows a drainage zone being constructed next to a battered concrete wall that is part of a spillway chute. In this application, perforated pipes in a gravel backfill are used to provide drainage behind the wall. Since the gravel drain is not filter compatible with the foundation, an intervening sand layer is used to provide filter protection. This is a two-stage system used to protect the foundation while providing drainage for the wall.

2.4 Foundation filter and drainage zones

The major types of foundation filters and drains included in the design guidance published by the major U.S. Government design agencies are described in following sections, with emphasis on their filter and drainage function(s). The interrelationship between these foundation elements and embankment filter zones are described in Section 2.3. Additional information on foundation drainage is provided here {Link_030}. Foundation dewatering may be required for installation of toe drains, and a brief description can be found here {Link_032}.

2.4.1 Blanket drains

Blanket drains may be included in embankment designs both to collect seepage from the foundation and to provide an outlet for seepage collected by a chimney filter/drainage zone. Since a blanket is at the interface between the embankment and foundation, it could be classified as either an embankment or foundation element.

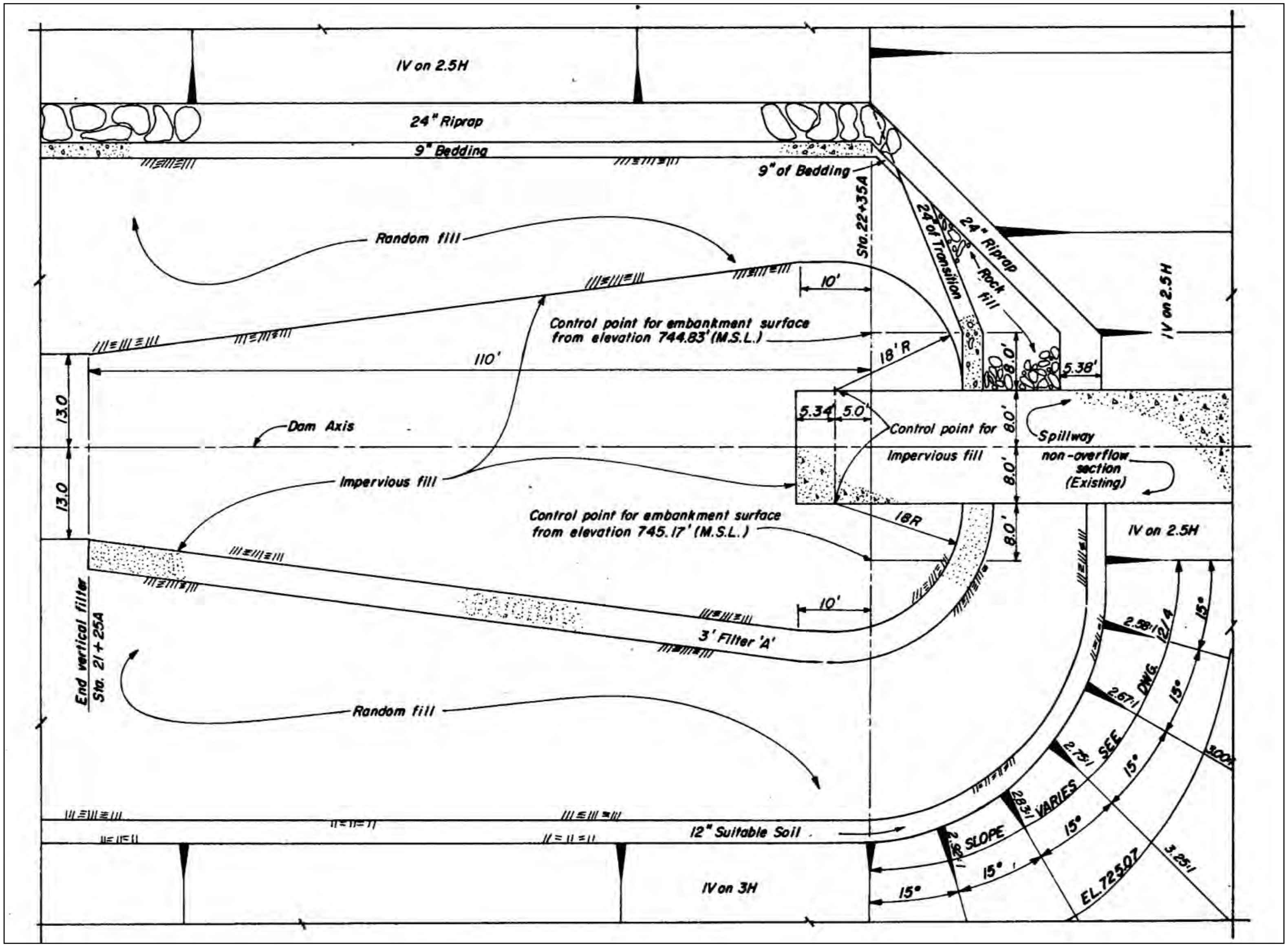

Figure 2-6. Filter protection used in the embankment section as it abuts the concrete section of a composite dam.

Figure 2-7. Filter and drainage zones to provide pressure relief and drainage of backfill next to training wall for a spillway chute. One individual is standing on top of the sand layer (Photo courtesy of NRCS, Texas).

Blanket drains must provide filter compatibility between foundation soils or bedrock that is not filter compatible with the overlying embankment. A properly designed blanket drain will protect finer embankment soils from piping into underlying coarser foundation soils or bedrock with joints and fractures as shown in Figure 2-8. It can also protect foundation soils from piping into a coarser overlying embankment zone.

Situations in which blanket drains are used:

- When the downstream shell is founded on a pervious sand and/or gravel foundation and the downstream shell soils are filter compatible with the foundation, a blanket is not required because the foundation

Figure 2-8. Pressure washing joints and fractures in bedrock prior to dental grouting and covering with blanket drain under downstream shell of dam. (Photo courtesy of NRCS.)

material effectively acts as a blanket zone. This configuration is independent of whether or not a chimney is used.

- If a chimney is included and there is no clear path for discharge, such as a sand and/or gravel layer as described above, a blanket drain or strip outlet drains must be included.
- Blankets are intended to collect foundation seepage and transmit any seepage collected by a chimney to the downstream toe drain. Blankets are not intended to control the phreatic surface through the dam since the core material will have a higher horizontal permeability than vertical permeability due to the material being placed and compacted in horizontal lifts. Interception of primarily horizontal seepage is achieved by a vertical drainage element, such as a chimney.

An example of a two-stage filter/drain blanket is shown in Figure 2-9 to 2-11. In this application, shown adjacent to an outlet works conduit, the first stage filter is placed on the foundation to protect against soil erosion caused by seepage flow from the foundation into the downstream shell. Over that layer, the second stage gravel layer is placed that provides

Figure 2-9. Fine filter being placed on the bedrock surface under the downstream shell of an embankment. View is toward downstream toe. Conduit is on the right of photograph. Exposed bedrock not yet covered is in background behind excavator. (Photo courtesy of NRCS, Alabama.)

drainage of the collected water to the downstream toe of the dam. Over that, another first stage filter is placed, which prevents erosion of the overlying shell into the blanket drain. This blanket then serves the purpose of protecting two seepage paths: one from the foundation and the other from the shell. Note that seepage through the shell can come from a phreatic surface that is not adequately attenuated by the chimney or by the precipitation that can percolate through the shell.

Additional discussion of the historical background of the use of blankets is described here {Link_030} and here {Link_033}.

2.4.2 Toe drains

Drainage trenches at the downstream toe of embankment dams, also known as toe drains, have been used in embankment dam design for decades. As with other types of filters and drains, the design and layout of toe drains has changed through time. These types of drains are most often constructed near the downstream toe of the embankment, although, in some applications, they are placed under the downstream shell

Figure 2-10. Gravel blanket drain being placed over fine filter shown in Figure 2-9. (Photo courtesy of NRCS, Alabama.)

Figure 2-11. Fine filter placed over gravel blanket drain shown in Figure 2-10. (Photo courtesy of NRCS, Alabama.)

{Link_031}, a practice that should generally be avoided since removal of the shell would be required if repairs are needed. The purpose of a toe drain is to collect seepage from two sources: the chimney/blanket drains and foundation seepage below the dam (underseepage). Toe drains placed on dam abutments will also collect abutment seepage. In any of these instances, the intercepted flow should result in a reduction of hydrostatic pressure under the dam and downstream of the toe.

Toe drains should consist of a perforated pipe surrounded by a gravel drain which, itself, is surrounded by a sand filter. This arrangement is known as a two-stage toe drain (see Figure 2-12). An example of a two-stage toe drain is presented in Attachment E, Case Histories – Narrow Toe Drain. While foundation conditions vary, this arrangement is considered

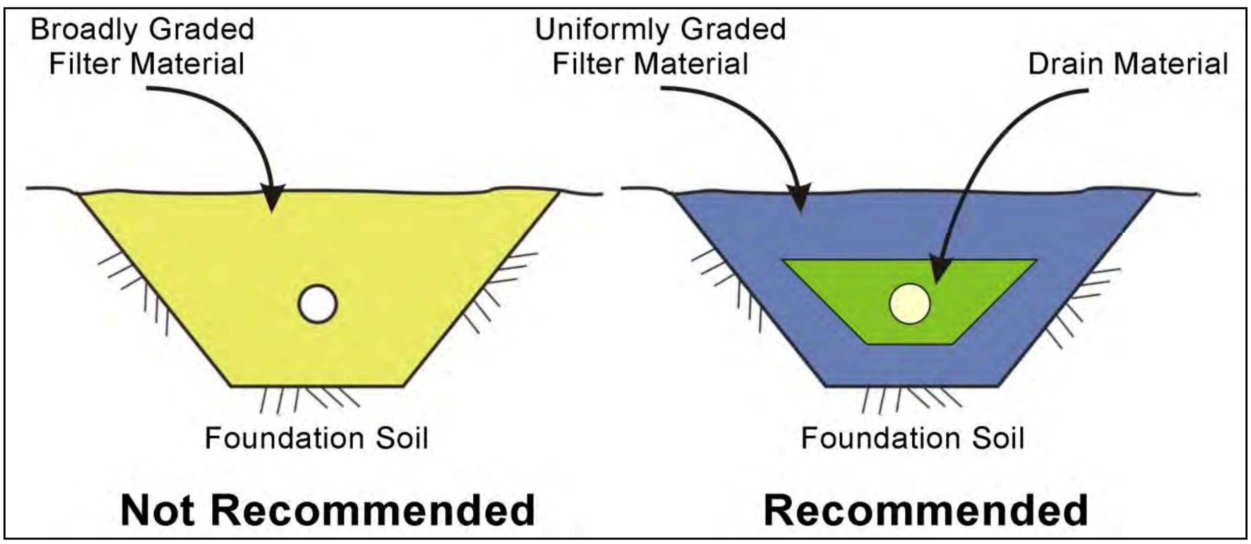

Figure 2-12. Typical one-stage (left) and two-stage (right) toe drains in a trapezoidal trench.

the minimum necessary for an effective drain. In the case of pervious foundations, the importance of collecting seepage and, more importantly, reducing pressure, cannot be overemphasized. For pervious foundations, it may be tempting to cut costs, and since drains are high-cost items, they may be the focus of such efforts. As described in Attachment A, such an approach can lead to a design that does not achieve the goal of pressure reduction and, in the case of modification to existing dams, can make the existing situation worse.

Single stage toe drains (a drain consisting of only filter sand and a drain pipe) may also be considered in the interest of minimizing costs. Single stage toe drains are not recommended due to potential uncertainties in foundation conditions and structure performance upon first filling.

While toe drains transfer and discharge seepage away from the dam, they also are important features for the monitoring of embankment dams. Monitoring of dams is important because as dams age, their performance will change. A design flaw or mistake made during construction can go undetected for years, or even decades, and monitoring will aide in the long-term performance of the structure. Toe drains permit three key features in such a monitoring program: flow measurement, detection of cloudy seepage, and sediment accumulation. All three can be achieved in an inspection well installed either at the discharge end of the toe drain or along the toe drain alignment. An inspection well generally consists of a flow measurement device (either a weir or a flume) and a sediment trap upstream of the measurement device. Details of toe drains and inspection well configuration can be found in Embankment Dam Seepage – Best Practices for Monitoring, Measurement, and Evaluation (FEMA 2009; Pabst 2007b).

As described in the following sections and FEMA P-676, toe drains can be constructed of several different geometries and construction methodologies. The type of configuration that is used is dependent on the expected amount of seepage. Two types of trench geometry used are rectangular and trapezoidal cross sections. Rectangular trenches with vertical side slopes are typically used where seepage is expected to be small. Trapezoidal trench sections are used where larger amounts of seepage are expected.

The potential for an increase in hydraulic gradient should be considered when toe drains are added to or replaced in existing dams. At sites where the piezometric head is near or above the ground surface, the addition of a toe drain will decrease that pressure. However, it should be noted that the differential head between the reservoir and downstream toe will increase. This increase in differential head may lead to an increase in hydraulic gradient through the foundation and subsequently increase the chance for particle movement over existing conditions.

Vertical versus trapezoidal trenches

As previously stated, toe drain trenches may be designed with either vertical sides as shown in Figure 2-13 or sloping sides as shown in Figure 2-14. Safety considerations will limit how deep a vertical trench can be excavated if it is required that construction workers and other personnel enter the trench. Trenches having vertical side slopes are less expensive since they require less excavation and processed backfill. Complications exist for the construction of two-stage toe drains in small spaces. One method used to eliminate such problems is the use of a "dog house" form that allows the introduction of the filter and drain material separated by a moveable form, as shown in Figure 2-13. Care needs to be taken that sufficient material is placed under the haunch of the pipe in order to provide adequate support.

Figure 2-13. Rectangular cross section foundation trench drain with gravel filter surrounding perforated collector pipe and fine sand filter in primary part of drain. Boxes are contractor's ingenious idea of placing the coarse filter around the pipe. By closing the top of the box, fine drain fill can be placed and kept separated from the coarse drain zone.

As indicated by the photographs in Figures 2-13 and 2-14, the trapezoidal cross section permits for a deeper toe drain installation and a greater surface area of drainage material for interception of water flow through the foundation. Therefore, the trapezoidal section will provide a more robust method of flow interception for sites with seepage concerns.

One-stage versus two-stage design

Historically, toe drains have incorporated one-stage and two-stage designs, as shown in Figure 2-12. One-stage designs were used when small

Figure 2-14. Trapezoidal foundation trench drain at toe of embankment.
Coarse inner filter surrounds perforated PVC collector pipe and
fine filter provides filter compatibility with foundation soils.

amounts of seepage are not expected. Two-stage designs are used when a large amount of seepage is expected. Incorporation of a perforated drainage pipe to facilitate flow is almost always done on a two-stage design. Collecting water in a toe drain system is not always easily accomplished, and attention should be paid to how water flows through the system (Pabst, 2007a). Additionally, design of filters placed on foundation soils is complicated by a greater variability of those materials than core material, or other engineered fills. Gradation of a toe drain should be checked to make sure the filter will not act as a barrier to any foundation units. Such barriers do not provide sufficient pressure relief, and in situations where an existing dam is being modified, the conditions can be made worse.

Collector Pipes

Collector pipes have a long history of poor performance in embankment dams. Earlier materials such as clay, concrete, and corrugated metal pipe have had poor strength and/or joint performance. Pipe junctions have also been an issue since no manufactured products existed during this era, and the junction was usually made by a "field fit." Figure 2-15 shows such a junction for a Y connection in clay tile pipe. Plastic pipe has also been used, and while its performance has been better, it has not been without its problems. Some polyvinyl chloride (PVC) products were brittle and did not withstand the rigors of heavy construction, and aging has been an issue with some high-density polyethylene (HDPE) products.

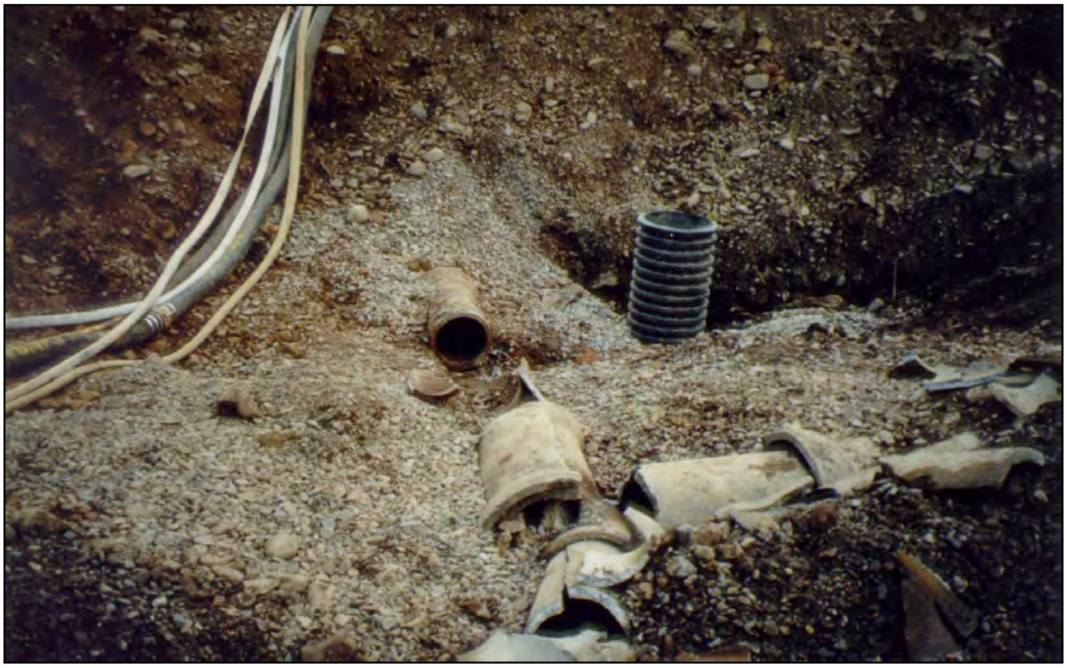

Figure 2-15. 1950s era concrete pipe used as a toe drain. Water enters the pipe through a gap left in the bell and spigot joints. A "Y" junction is shown with two laterals that connect to a trunk line shown on the right side of the photo. Since connectors were not manufactured for this configuration, intact pieces of pipe were broken and the pieces used to stack together, making a protective cap for the junction. This junction was exposed during excavation for a toe drain replacement.

In the last two decades, corrugated HDPE pipe was a popular choice for Reclamation toe drain construction. In the late 1990s, video examination of Reclamation toe drains showed that a number of these installations were exhibiting some form of distress, ranging from minor deformation to complete collapse. Most of these cases were single-wall corrugated HDPE, which has been found to experience strength loss with time. Due to the high number of structural failures and lack of laboratory data on the strength of perforated versus non-perforated plastic pipe, Reclamation undertook a study to evaluate these products (Reclamation, 2009). That study found that perforated corrugated pipe (PVC or HDPE) had the same load carrying capacity as non-perforated pipe since the strength of the pipe comes from the outside corrugations, which are not perforated. The study

also demonstrated that perforated solid pipe has a diminished strength in relation to non-perforated pipe and showed that some PVC products are brittle. The report also addressed installation issues, commonly available perforation sizes, and joint types for the different products. Since failure of pipes designed based on static conditions (overburden) has occurred, it may be that construction loads are the more critical loading condition.

Joints for corrugated HDPE and PVC pipes are typically bell and spigot or butt joint with a collar. While gaskets are available for most of these joint types so they are watertight, the greater concern is proper field installation. If pipe ends or couplers are damaged or get dirty prior to connection, marrying the pipe segments can be difficult. Frustrated workers may struggle with a pipe connection and give up prior to the joint being completely closed. Recent video inspections have shown that poor joint connections are as much of a problem as crushing in the center section.

Taking these factors into account, profile HDPE pipe is recommended for use in toe drain applications. The advantages of this pipe type over all others are:

- Large load carrying capability.
- When a load carrying capability much greater than that needed for overburden is used, the pipe is more likely to withstand poor or incorrect installation methods.
- Joints are welded, strong, and water tight.
- Junctions are factory welded, strong, and water tight.
- Aftermarket perforations can be used allowing the designer to specify the perforation size permitting more flexibility in the selection of gravel envelope material.

Perforated collector pipes should always be inspected by video camera at the end of construction to verify no damage occurred during installation. Historically, a second method has been used to inspect toe drain pipe that consists of pulling a ball or torpedo-shaped object through the pipe. While this method can be used, it should not be the sole source of installation acceptance since the method is easily bypassed if not visually monitored.

Most perforated collector pipes will have some amount of material in the pipe invert or contain some kind of clogging in the perforations consisting of algae, roots, or sediment. Since power washing is now commonly available, it is possible to flush out such pipes. Before doing so, consideration should be given whether the pipe will be damaged or an erosion condition aggravated. If the drainage system design is of high quality, then cleaning can be used. If the drains are of poor or unknown quality, cleaning should be avoided since the system may have “self-healed” to a stable condition, and cleaning it could reactivate material movement.

2.4.3 Relief wells

In a foundation where a pervious layer is overlain by an impervious layer (or stratum), the pervious layer may contain high pressures or artesian conditions. This can lead to heaving (blowout) of the overlying impervious layer (aquaclude). In these situations, it may be impractical to construct a toe drain down to the pervious layer, especially if it is a significant depth (> 20 ft). In such cases, pressure relief wells can be used. Relief wells are constructed with well screens, much like a water well, with an annular space surrounding the well screen containing a designed filter pack. Relief wells are usually outletted to the ground surface. A more detailed description of relief well design, construction, and maintenance is provided in EM 1110-2-1913, “Design and Construction of Levees (USACE, 2000), EM 1110-2-1914, “Design, Construction, and Maintenance of Relief Wells” (USACE 1992), and NAVFAC DM-7. It should be noted that the particle retention criteria for well design may differ from what is presented in this manual. Typically, well design criteria are more strongly influenced by permeability requirements.

Relief wells have a distinct disadvantage in that they require ongoing maintenance to rejuvenate their flow capacity. Iron ochre and chemical incrustations are a plague to relief wells, and the cost to maintain their capacity must be factored into a life cycle cost for relief wells. Due to this maintenance issue, as well as the ineffectiveness of wells intercepting 100% of foundation flows, toe drains are the preferred pressure reduction measure.

2.4.4 Slurry trench filters

As described previously, when drainage or filtration is required at the downstream toe of a dam, a high water table or confined aquifer can make filter/drain installation difficult. Another method used to install a filter or drain is the slurry trench method. The use of a slurry trench seems counterintuitive since slurry trenches are often used to construct cutoff walls through dams. The use of a bentonite slurry is also contrary to constructing a drainage element that provides high permeability relative to the surrounding foundation. To overcome these obstacles, a slurry trench method was developed using degradation technology (Fisk et al., 2001 and Gerhart et al., 2005). In this method, a synthetic biopolymer or other organic admixture, such as guar gum, is used in place of the bentonite admixture used in more common slurry applications. These admixtures are mixed with water to produce a slurry that stabilizes the trench long enough to place the filter or drain backfill. Biodegradation of the slurry then occurs, permitting the trench to act as a flow interceptor.

2.4.5 Modification of existing drainpipes

Many existing dams have seepage issues related to misunderstood site conditions, poor design, poor construction techniques, or a combination of all three. Adding to these problems can be the inclusion of improperly designed drainage features. For several decades, toe drains consisted of butt joint pipe surrounded by coarse gravel as shown in Figure 2-16. The gravel seldom met particle retention criteria for the foundation soils, and separation between the pipe joints was seldom controlled, thus permitting passage of finer grain soil through the gravel filter. These conditions have resulted in active piping through the drainage system.

Additionally, older drainpipes do not have sufficient strength and may be cracked, deformed (see Figure 2-17), or completely collapsed. When the pipe begins to fail, this leads to greater amounts of material entering the pipe and rendering many systems completely clogged with foundation material as shown in Figure 2-18.

Since many toe drain installations were installed with no consideration given to future examination, video investigations can be complicated. If pipe elbows and bends were typically installed, video cameras may not be able to get past those points. Also, if the drain was clogged with material,

Figure 2-16. Clay tile pipe surrounded by gravel-size material. Note mechanical pencil for scale. Surrounding the gravel is a mixture of silt and sand backfill that does not meet filter criteria for the gravel. Seepage enters the pipe through joints between pipe segments. The silt and sand can erode through the gravel backfill and enter the pipe through the joints.

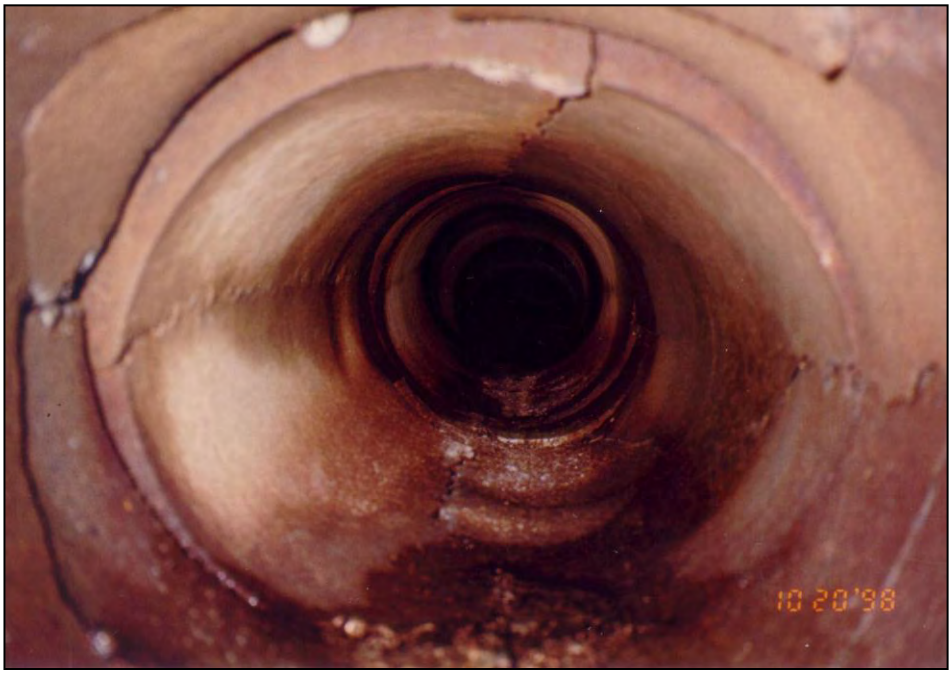

Figure 2-17. Interior view of a reinforced concrete pipe from the 1950s. Note that the pipe is overstressed, and cracks have formed at the crown and spring line. The pipe has also deformed to an oval shape. In the foreground, a joint can be seen and sand that passed through the joint.

Figure 2-18. Clay tile pipe from 1916 as it was exposed during excavation. Note that the pipe was completely clogged with silt and sand.

or crushed, examination is not possible. Vegetation could also lead to problems with existing drainpipes. As a concentrated source of water, drains are attractive to plant roots. In extreme cases, root growth can completely clog a pipe, greatly reducing its flow potential as shown in Figure 2-19.

Figure 2-19. During modification of a dam, this toe drain pipe was exposed during excavation. The pipe was completely clogged with the root ball shown in the foreground. It was noted that a tree was growing over the toe drain, and the drain was probably the water source in this arid region of central Oregon.

Typically, a deficiency is identified for clogged pipes and a construction effort is undertaken. Repair of existing drains is uncommon, and total replacement is the more usual course of action. When replacing existing drains, consideration should be given to the amount of flow collected by those drains. While the pipe itself is in poor condition, and particle criteria are not met, these conditions can result in attractive interception of groundwater flow at the expense of particle retention. Replacement with drains not meeting particle retention criteria can result in significantly less interception of seepage. This, in turn, can result in higher pressures and possible seepage discharge from the ground surface—a situation that did not occur prior to the repair. Attachment G describes these cases and how to address them.

2.5 Recommendations

- For design of new dams, filter diaphragms are required around all conduits that pass through embankments regardless of embankment height, site conditions, or hazard classification.
- Full filter protection is recommended for dams that are classified as significant to high hazard potential.
- Cost should not be the sole basis for eliminating filter protection in small dams.
- When modifying an existing dam, filter protection needs to only be added for identified deficiencies or potential deficiencies that are judged to pose unacceptable risks.
- Existing dams with large amounts of seepage can be made worse by the addition of a new toe drain that is less permeable than more pervious foundation layers. Such drains will act as barriers to more pervious seepage paths and lead to an increase in pressure.
- Relief wells clog with time, resulting in diminished effectiveness. A routine schedule of cleaning and pump testing should be part of the relief well operations and maintenance requirements.

3 Additional Applications

3.1 Abandonment of old drains and grouting

Drainpipes, as referenced in this document, are structural pipes used to convey seepage water collected in a toe drain system to a discharge point downstream of the dam. The materials used for these pipes have changed through history. Early dam construction typically used rigid pipe (clay tile, cast iron), with flexible plastic pipe becoming more popular since the 1980s.

Additionally, corrugated metal pipes were commonly used in collector systems, but the experience with deterioration and subsequent piping of surrounding filters into the pipes caused these materials to be regarded as a poor choice for a collector pipe. Asbestos cement pipe was also historically used in many collector systems, but the hazard from asbestos has led to its discontinued use.

Many drainpipes in older structures are in poor condition. Causes of this poor condition include deterioration, improper design, damage during installation, and post-construction damage. Many dam safety deficiencies are associated with the poor condition of these pipes.

The integrity of drainpipes should always be evaluated during dam safety inspections. One of the most effective examination methods is to perform a video examination of the pipe (Federal Emergency Management Agency [FEMA] 2007). These investigations may discover that material has eroded into the pipe, the pipe is deformed and cracked, joints are offset, or the pipe has collapsed. This information can indicate a larger dam safety issue that requires structural modification. The issue then becomes how to deal with these pipes during the modification.

Ideally, all damaged or incorrectly designed pipes would be removed during the modification. This is not always practical, and this section will describe what steps can be taken to address these pipes and how those steps relate to any filters that may be installed during the modification.

Access to existing pipes is typically problematic since access features were seldom incorporated into the original design. Additionally, if drainpipes have changes in alignment, it will be difficult or impossible to get cameras or repair tools past these bends even if the pipe is in good condition. An excavation is then required into this area to gain access to the entire pipe length. If the drainpipe was installed near the interior of the dam, or a raise was added to the embankment, these excavations can become deep.

In other cases, installation of a replacement drainage system removes some, but not all of the existing drain pipe. In either case, there are two methods to address drainpipes that are not completely removed: slip lining or grouting. Slip lining consists of inserting a plastic pipe inside the existing pipe that will take over the particle retention properties that are missing in the existing pipe. As may be expected, depending on the condition of the existing pipe, this can be difficult to execute in the field. Suggested guidance for this method follows.

1. Examine the interior of the existing pipe using a video camera. This should be done prior to specification preparation. If the situation requires that it be done during construction, the specification requirements for the contractor will need to have enough flexibility to ensure modification objectives are met without imposing a large amount of monetary risk to the contractor.
2. Determine the foundation grain size distribution and calculate the perforation size of the replacement pipe using these data.
3. Determine the size of the replacement pipe. The diameter of the replacement pipe should not be greater than one-half the diameter of the existing pipe, although experience has shown that sizes even smaller than this are required for practical reasons. Also note that sags or deformations and joint offsets in the existing pipe can result in a reduced effective diameter. It may also be prudent to have several pipe sizes available during construction to offer flexibility during installation. The wall thickness of the pipe should be designed for the potential installation stresses and strains (see the next step).
4. Determine the method of advancement for the replacement pipe. The replacement pipe can be installed by pushing or pulling. If there is access to only one end of the pipe, the liner will have to be pushed (deadheaded). If both ends are open, a fish can be sent through to attach to the replacement pipe, and it can be installed by pulling from one end and pushing

from the other. For either method, a special front piece (torpedo) should be utilized that will assist in guiding the liner through the existing pipe. Note that even with ample clearance, the weight of the liner, coupled with drag, will make unaided advancement impractical. If material is present and begins to move within the annular space, even greater force will be required. Therefore, some type of machinery will be needed for advancement of the pipe. Such equipment includes utility tractors for pushing or pulling and winches for pulling.

The second method, grouting, consists of placing a cement-based grout into the existing pipe. It is recommended to grout the entire length of existing pipe since leaving some sections open can result in later ground subsidence if the pipe were to collapse. The grout should never be injected into the pipe, but rather it should be placed by using the slick line method. In this method, a grout line (slick line) is inserted to the far end of the pipe (assuming access is from only one end) to be grouted and a temporary plug placed over the open end. A hole is then made in the top of the plug. Grout is introduced into the slick line and fills the pipe until it reaches the hole in the top of the plug. Once grout comes out the hole, the slick line is retracted while still supplying grout to the pipe. A calculation should be made of the pipe volume and compared against the grout take. If the grout take is less than the pipe volume, the pipe has not been completely grouted. If it is greater, there was grout intrusion into the foundation. It should be noted that the purpose of the grouting is to fill the pipe, not to grout the foundation. For this reason, thicker grouts should be used at low pressure. Additionally, grouting should never be done after new filter or drain materials have been placed due to the possibility of grout intrusion into those materials. Grouting operations should be completed prior to foundation acceptance, which is then followed by fill placement. Note that neither of the above methods eliminates the chance of having an internal erosion failure. The best choice is simply to remove or replace the pipe.

3.2 Adding filter protection to existing conduits

Many existing dams, both large and small, were originally constructed with outlet works or other conduits without filter protection. If a dam safety issue has arisen due to poor performance of an existing conduit, or a chimney filter is being added to an existing embankment, adding a protective filter to the conduit may be warranted. This section will focus on outlet works or other types of conduits, such as spillway conduits, that were constructed and then covered with embankment fill. These conduits are typically constructed in one of two ways: (1) cut and cover if they are constructed below existing grade and (2) at-grade if they were built on the existing ground surface.

Conduits on soil foundations require filter protection around the entire conduit. Exposing a conduit and adding a filter to only the sides and top will leave the foundation under the conduit unprotected. Piping channels can form under conduits since the conduit may act as a roof for the piping channel. A reliable method for filter placement under a conduit is also needed since any gap or low density areas will render the protection useless. Some methods have been proposed for addition of a filter under a conduit that are considered unacceptable. Those methods are summarized in Table 3-1.

Table 3-1. Unacceptable methods for adding filters under conduits.

Method	Discussion
1. Excavating under half of the conduit and backfill with filter material. Next, excavate and backfill under the other half.	Filter material cannot be compacted sufficiently to prevent settlement once the water table rises.
2. Cutout a section of conduit floor, place filter, replace floor.	Since reinforcement will be cut in reinforced concrete conduits, the hoop strength of the conduit will be lost.
3. After placing the filter using one of the above methods, grout from inside the conduit to fill any voids between the bottom of the slab and top of the filter.	Grouting operations should never be carried out adjacent to filters since they can become contaminated with grout, rendering the filter useless.

In the interest of providing intimate contact between the filter and the bottom of the conduit, a section of the conduit should be removed and re-constructed after filter placement.

3.2.1 Location of filter around conduit

The preferable location for adding a protective filter around existing conduits is near the centerline of the dam, but locations near the downstream toe are also acceptable. The centerline location is preferable since the greater overburden stress will provide greater confining stress that will keep the filter in contact with the conduit and will have greater resistance to hydraulic fracturing. Adding filter protection near the centerline of the dam will require removal of a significant portion of the embankment, including the crest. If reservoir operation is to be maintained during construction, this method may not be acceptable. A cross section of a typical filter addition near the centerline of a dam is shown on Figure 3-1.

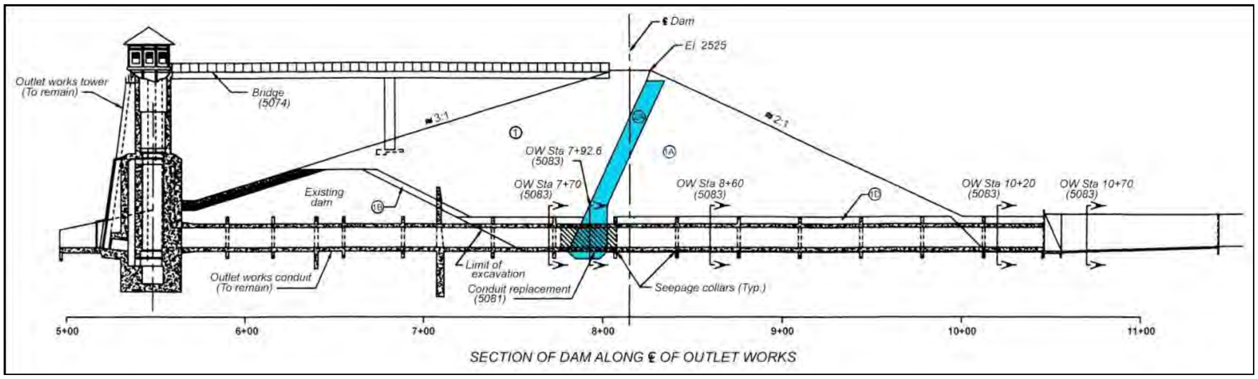

Figure 3-1. Typical filter addition around a conduit near the centerline of a dam enlargement.

Diaphragms can also be added to downstream locations, but sufficient overburden is required to overcome any "blowout" concerns. Assuming a seepage path exists along the existing conduit and full reservoir head is expected at the filter diaphragm, sufficient overburden is required to overcome the hydrostatic pressure. This can be accomplished by placing a stability berm at the downstream toe over the filter diaphragm. Assuming the density of the berm is twice the density of water, the berm height should be one-half of the reservoir height. A cross section of a typical filter addition near the downstream toe of a dam is shown on Figure 3-2.

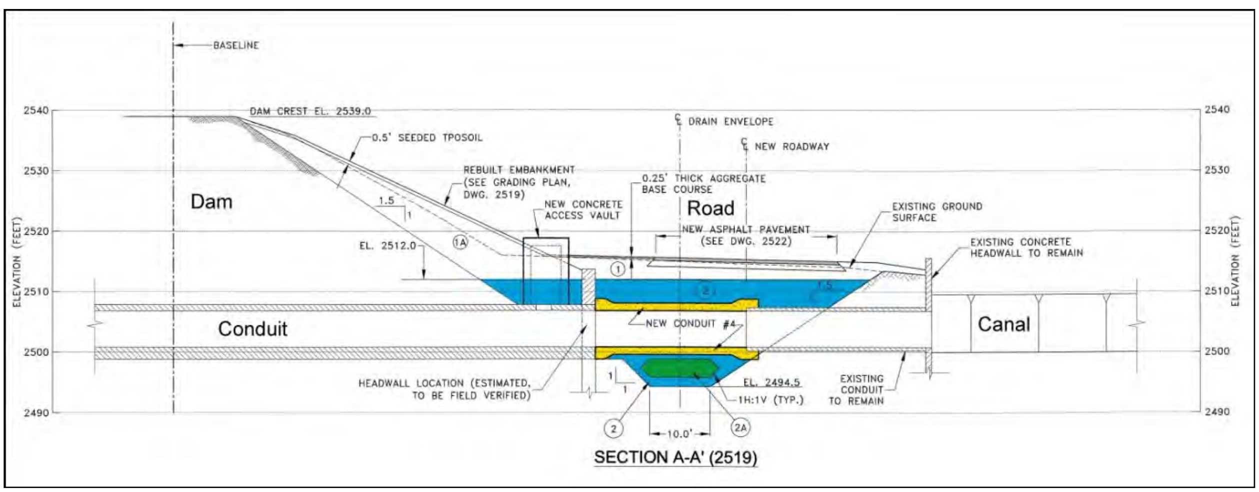

Figure 3-2. Typical filter addition around a conduit near the downstream toe of a dam.

Acceptable construction methods for the addition of a filter diaphragm around an existing concrete conduit are included in Table 3-2. The procedures would be similar for other conduit types, although the addition of a cradle may be required.

Table 3-2. Acceptable method for addition of a filter to an existing conduit.

Step 1	—	Excavate around the conduit, exposing it in the area of filter placement.
Step 2	—	Sawcut through the conduit and demolish between the sawcuts.
Step 3	—	Excavate into the foundation under the conduit profile a minimum of 2 ft. The trench width (measured upstream to downstream) should be greater than 6 ft. The upstream and downstream side slopes should be 2H:1V or flatter. An offset of at least 1 ft should be used between the top of the excavation slope and the sawcut face.
Step 4	—	Inspect and accept foundation. Proof roll the foundation.
Step 5	—	Place the filter material in the bottom of the trench and compact. Check the filter density with an in-place density test.
Step 6	—	Re-build the conduit.
Step 7	—	Replace fill, including filter diaphragm around conduit. Construct stability berm, if required.

3.2.2 Minimum dimensions for filters added to existing conduits

The minimum dimension for the addition of filter protection around existing conduits is a function of the conduit size and whether or not seepage collars are present. For conduits that do not include seepage collars and have an inside diameter of 2.5 ft or less, the guidance given in FEMA 484 can be considered. In that guidance, filter protection extends three pipe diameters around the sides and top of the conduit and 1.5 pipe diameters below the conduit. The filter thickness (measured upstream to downstream) should not be less than 3 ft. Since piping failure modes along conduits are based on flow along the outside of the conduit, the above rules should be based on the outside or maximum structural dimension. If the pipe is encased in concrete, or the pipe is set in a concrete cradle, the outside dimension of the concrete should be used.

For conduits larger than 2.5 ft outside diameter, that do not include seepage collars, the minimum extent of filter protection should be at least 8 ft for the sides and top and 4 ft under the conduit. The filter thickness (measure upstream to downstream) should not be less than 8 ft.

For existing conduits that include seepage collars, regardless of conduit size, the extent of filter protection is defined by the size of the collar. In these cases, the filter extent should be not less than 8 ft beyond the limit of the sides and top of the seepage collar. The filter should extend no less than 4 ft below the bottom extent of the collar. The intervening space between the outside of the conduit and the outside edge of the seepage collar should also be filled with filter material.

Example: A 6-ft inside diameter reinforced concrete conduit has an exterior horseshoe shape. The lateral external structure width is 8 ft. The structure includes seepage collars that extend 4 ft beyond the outside shape of the structure. That is, the extent of the seepage collars mimics the outside shape of the structure on the top, sides, and bottom. For this case, a diaphragm filter with the following dimensions would be used:

Side	Extent beyond seepage collar: 8 ft
	Extent beyond side of structure: 8 + 4 = 12 ft
Top	Extent beyond seepage collar: 8 ft
	Extent beyond top of structure: 8 + 4 = 12 ft
Bottom	Extent beyond seepage collar: 4 ft
	Extent beyond bottom structure: 4 + 4 = 8 ft

3.3 Geotextiles in embankment dams

The following statement explains the current practice for using geotextiles in U.S. dams. The statement is taken from the July 2007 draft of "Geotextiles in Embankment Dams," *Status Report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation*:

> *Geotextiles are used in a variety of applications in embankment dam construction and rehabilitation. Although policy varies, most practitioners in the United States limit the use of geotextiles to locations where there is easy access for repair and replacement (shallow burial), or where the geotextile function is not critical to the safety of the dam should the geotextile fail to perform.*

> *In a limited number of cases, geotextiles have been used as deeply buried filters in dams in France, Germany, South Africa and a few other nations. Most notable, is a geotextile installed as a filter for Valcross Dam which has been successfully performing for over 35 years. These applications remain controversial and are not considered to be consistent with accepted engineering practice within the United States. Because geotextiles are prone to installation damage and have a potential for clogging, their reliability remains uncertain. Many organizations forbid their use in embankment dams in critical applications where poor performance could lead to failure of the dam or require costly repairs. Designers are cautioned to consider the potential problems associated with using a geotextile as a critical design element in a non-redundant manner deeply buried in a dam.*
>
> *It is the policy of the National Dam Safety Review Board that geotextiles should not be used in locations that are both critical to safety and inaccessible for replacement.*

The authors of this manual concur with this policy, and additional discussion is provided in the following section.

3.3.1 Technical evaluation of geotextile use in filter/drainage systems for dams

Sand and gravel filters have been tested in research studies simulating conditions within a dam and have been successfully used for many years as the main feature of filter/drainage systems to prevent piping and concentrated leak development in dams. This testing and extended successful use has demonstrated that the intended performance of these materials as filters for dams has been met. This is not the case with geotextiles as their usage in embankment dams has been very limited. It is useful to consider the characteristics of sand filters in evaluating their success and to compare these characteristics with geotextiles for determining whether geotextiles can provide the same desirable performance.

Clean sand or sand and gravel mixtures act as a cohesionless material. When there is very little or no binder material (fines such as silt and clay or a cementing agent) within the sand, it will flow to a soil boundary such as the side of a trench or a soil zone in an embankment and apply a positive pressure. The soil boundary acts as a barrier or containment for the sand as it is placed and compacted in a zone or trench. With no soil binder or cementing agent, the sand will shift or cave to maintain a continuous, homogeneous zone without cracks or openings as the dam settles or shifts during construction or during the first filling of the reservoir or an earthquake.

For intergranular seepage flow (seepage through soil with no cracks or defects), filters designed using current criteria were successful in testing studies for preventing any particles from detaching on the discharge face under high gradients. Apparently, there is some arching between the closely spaced contact points where the filter is in contact with the discharge face to prevent any movement of particles. Testing and experience shows that too coarse filters or other materials that do not support the discharge face with closely spaced contact points as seen in granular filters will not prevent soil particles from detaching when the seepage gradients exceed the critical gradient of the soil.

Geotextiles by themselves do not apply a positive pressure to the surface against which they are placed, as shown in Figure 3-3. Since the geotextile is a flexible fabric, it must have a material placed on the downstream side of the fabric to hold it against the discharge face. The material on the downstream side would need to be configured so that the contact points on the discharge face have similar spacing as the sand filter contact points.

Grid materials or gravels placed on the downstream side of geotextiles will not provide the proper support to the discharge face, and contact points will be too far apart to prevent soil particle detachment. The geotextile will bulge out away from the soil surface between the points where gravel particles are in contact. If seepage gradients just upstream of the geotextile exceed the critical gradient for the base material in the dam, soil particles will be detached from the face and soil in suspension will arrive at the geotextile face. For geotextiles designed with an apparent opening size (AOS; the equivalent opening size, EOS, was used before about 1993) to meet the filtering requirements of the soil, the particles in suspension will be caught at the filter face in a layered filter cake with a very low permeability. The result will be clogging of the geotextile at all locations where high gradients exist (usually large segments of the drain). For fabrics with a larger

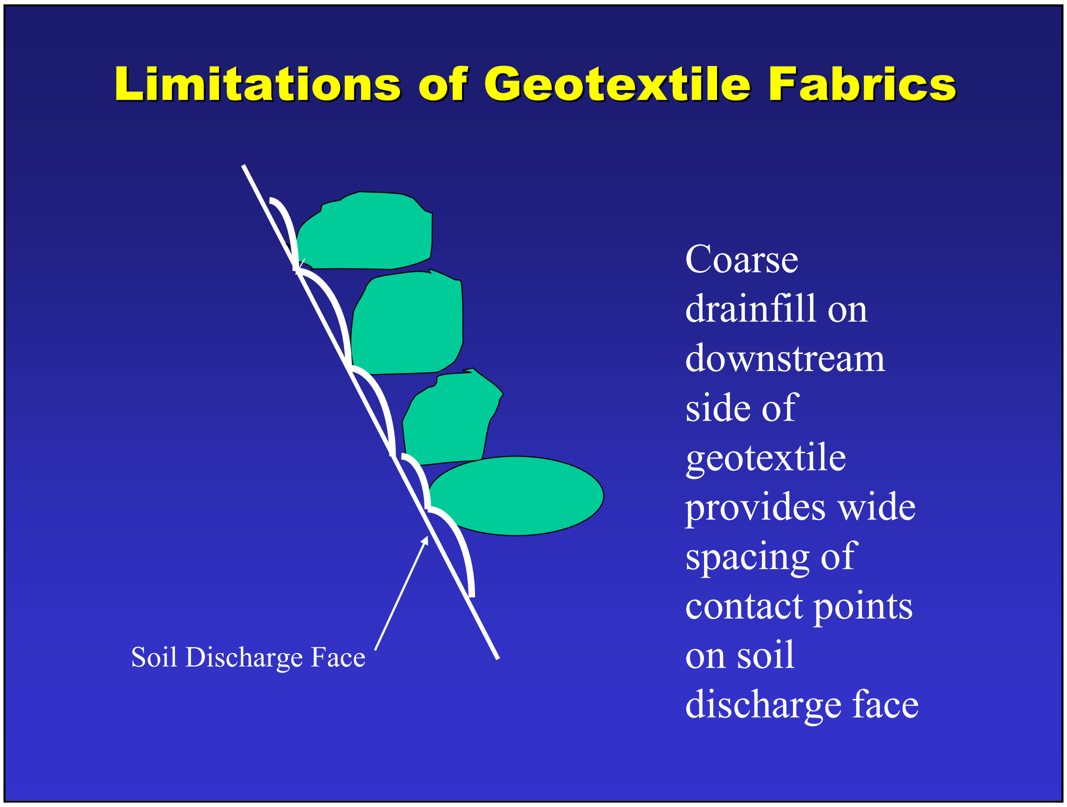

Figure 3-3. Cross section of a base soil covered by a geotextile that is then covered by coarse gravel. Due to the voids in the gravel, the geotextile can "flex" into these voids, resulting in the loss of positive pressure on the base soil discharge face. Base soil particles can then detach and clog the geotextile.

AOS, the soil will pass on through the geotextile, and a piping feature will develop in the dam and progress toward failure.

This condition is exhibited in the gradient ratio test performed on geotextiles. In this test, water under pressure is applied to a soil specimen that has a geotextile placed under it. Pea gravel is used to support the geotextile. In most cases, at least some clogging and/or passage of soil material through the geotextile is reported in the test results. For the cases cited (Giroud 2005) where geotextile use in dams has been successful (such as Valcros Dam), the seepage gradient may not be sufficient to cause removal of soil particles. Apparently no instrumentation has been installed to check the gradients in Valcros Dam or other dams cited where geotextiles have been successful, as these data are not given to support the performance. The only evidence given for these successes is that the dams appear to be performing well based on visual observation at the surface. It is possible that a given dam may be successful using a geotextile as the filter for the drainage system if the gradients remain low; however, most dams have the potential for high gradients that will cause particle detachment at the drain/soil interface. Also, piping/internal erosion is time dependent and may take more years to manifest itself visually.

There are many examples that demonstrate geotextiles do not prevent detachment of soil particles at the drain/soil interface when critical gradients are exceeded. Geotextiles used under riprap on the Tennessee-Tombigbee Waterway (U.S. Army Corps of Engineers, Engineer Technical Letter No. 1110-2-286, "Engineering and Design Use of Geotextiles Under Riprap," dated 25 July 1984) showed that if the AOS was too small, clogging of the geotextile was a problem, causing buildup of seepage pressure under the riprap. This clogging could happen only if soil particles were detached with seepage water flowing out of the channel bank behind the geotextile. Using a larger AOS would allow the soil particles to pass through the geotextile, but would then cause a potential piping problem. This may not be serious for a channel with riprap, but would be very serious for an earth dam that retains a large reservoir of water serving as an essentially infinite source of seepage water to develop a piping failure condition.

Most studies and reports on using geotextiles for highway drainage work indicate that geotextiles either clog or allow soil particles to pass through. The most significant of these is Geosynthetics Research Institute paper (GRI-18, "Rapid Assessment of Geotextile Clogging Potential Using the Flexible Wall Gradient Ratio Test," by T. D. Bailey, M. D. Harney, and R. D. Holtz) presented at the Geo-Frontiers Conference, 2005. The results cited in this paper indicated that most tests showed some to major clogging while other tests showed particles passing through the geotextile. While this may be acceptable for highway drainage, it is not acceptable for earth dam drainage. Additional reports showing similar results are ASTM STP-1281, "Recent Developments in Geotextile Filters and Prefabricated Drainage Composites," and NCHRP Report 367, "Long-Term Performance of Geosynthetics in Drainage Applications."

3.3.2 Historical use of geotextiles in earth dam construction

Geotextiles have been used as a separator between a sand filter and coarser fill in downstream toe drains. As long as a properly designed sand filter is placed next to the soil where high gradients may exist, the soil fines will be prevented from migrating to the geotextile where they could clog it. A geotextile will perform a separation function if it is located between two dissimilar soils or between a soil and a manmade material to prevent the mixing of the two materials and not as a filter/drainage function. However, caution should still be exercised since even a small amount of fines in the filter can clog the geotextile. For this reason, this arrangement is not recommended.

There have also been successful drainage applications of geotextiles used in trench drains away from the dam where the potential for high gradients is very low. In these applications, the geotextile has been placed next to the soil in a trench with a coarse gravel drainfill inside the geotextile with or without a perforated or slotted drainpipe to carry the seepage water to a safe outlet. In these successful cases, the seepage passing through the soil does not have a gradient large enough to detach the soil particles where the geotextile is not in intimate contact with the soil between the gravel particles. It is recommended that this design not be used due to the difficulty in determining the gradient at the drain and especially estimating what the critical gradient will be.

3.4 Recommendations

- Due to issues with clogging, geotextiles should only be used in non-critical areas of embankment dams.
- Existing drains, when abandoned but not removed, must be sealed to prevent the chance of any material eroding into damaged or poorly constructed drains. A sealing procedure, such as grouting, should not contaminate any new or existing filters or drains.

4 Laboratory Testing

4.1 Laboratory testing for particle retention

As described in chapter 1, laboratory studies have been used historically to obtain empirical relationships related to soil particle retention {Link_020}. This chapter summarizes and compares these test procedures. Complete test descriptions are included in Attachment F.

Experiments on filter compatibility for silts and clays were reported by Sherard (1984) in an American Society of Civil Engineers article, "Filters for Silts and Clays." First, intact specimens of silt and clay that were from 30 to 60 millimeters (mm) (1.18 in. to 2.36 in.) thick were compacted against filters, some of which were significantly coarser than filter criteria would require to protect the base soils against piping. The tests began with hydraulic gradients in the range of 167 to 333. At these gradients, failures in the base specimen could not be induced because the discharge energy was insufficient to initiate piping. Only when applied gradients were increased and hydraulic fracturing was induced were failures initiated. Based on these tests on intact clay base specimens, researchers developed an alternative test that used a preformed slot or hole in the base soil {Link_021}. This was preferred to allowing the specimen to hydraulically fracture because the flow path could be defined more precisely and studied in more detail. The early experiments used a slot with dimensions of about 12 mm × 1.5 mm. The length of the base soil specimen was about 6.5 in. (165 mm), and the filter section was about 3 in. (76 mm) long.

Outgrowth of this testing was the development of what is now called the No Erosion Filter (NEF) Test. The following summary and conclusions from Sherard's (1989) paper, "Critical Filters for Impervious Soils," explains the change in experimental apparatus. Figure 4-1 illustrates the concept.

1. *The NEF test is the best available test for evaluating critical filters located downstream of impervious cores in embankment dams. This is considered the most valuable single conclusion from the four-year long research effort. The*

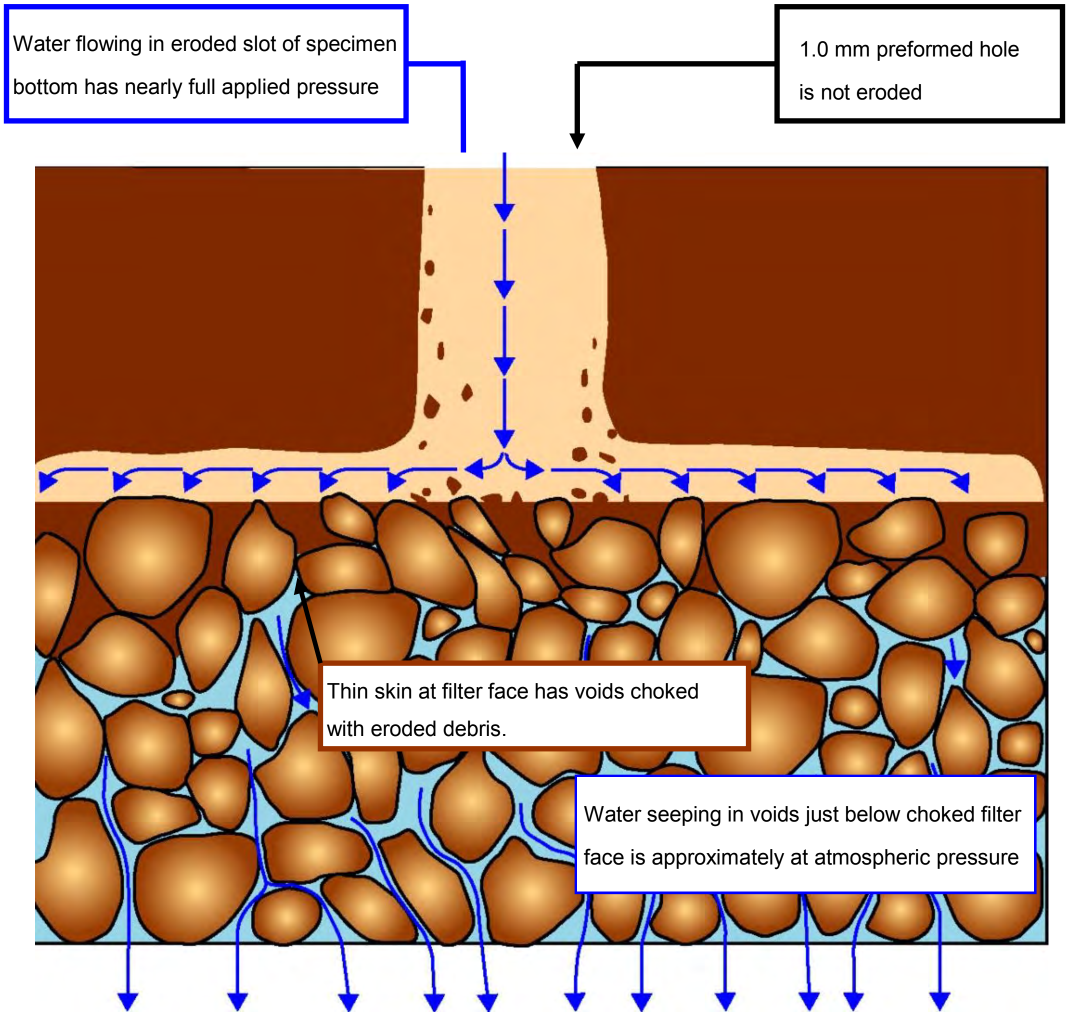

Figure 4-1. This sketch illustrates how a filter seal develops as eroded particles are carried from the sides of a crack in the base soil to the filter face. Eroded particles accumulate and create a filter seal that effectively blocks further flow and subsequent particle movement (after Sherard, 1984).

conditions in the test duplicate the most severe conditions that can develop inside a dam from a concentrated erosive leak through the core discharging into a filter. For tests with filters finer than the filter boundary (D15 smaller than D15b), there is no visible erosion of the walls of the initial preformed leakage hole passing through the base specimen.

2. *The NEF test is a simple test that can be made in any soil mechanics laboratory. It gives reliably reproducible and easily interpreted results, and it is well adapted for testing the entire range of impervious soils used for dam cores.*

3. *The filter boundary D15b separating successful and unsuccessful tests for a given impervious soil, as determined by the NEF test, is unique. The boundary D15b is independent of the dimensions of the laboratory apparatus and is dependent only on the properties of the protected impervious soil (base). The filter boundary D15b can be considered a property of the base soil in the same sense that results of tests to determine the Atterberg limits and effective shear strength parameters are considered properties of the impervious soil.*
4. *Based on the results of NEF tests, soils used for the impervious sections of embankment dams fall into the four general categories shown in Table 1 depending only on fine content.*

The NEF Test apparatus and procedures are described in an article by Sherard, et al.(1985). A schematic of the test is reproduced in Figure 4-2.

The Natural Resource Conservation Service (NRCS) procedure for performing this test is reproduced in Attachment F. The Bureau of Reclamation (Reclamation) developed a standard test procedure for the NEF Test utilizing holes instead of a slot, and their procedure is referenced in Attachment F.

4.1.1 Continuing erosion filter test

Foster and Fell (2000) presented a modification to the NEF Test known as the Continuing Erosion Filter (CEF) Test. They recommended evaluating an existing embankment filter differently than when designing a new filter. The following quote is from their article:

> *An assessment of existing filters should consider how the filter may perform in the event of a concentrated leak developing through the core. The performance of filters in dams is classified into three categories as follows:*
>
> - *Seal with no erosion-rapid sealing of the concentrated leak, with no potential for damage and no or only minor increases in leakage*

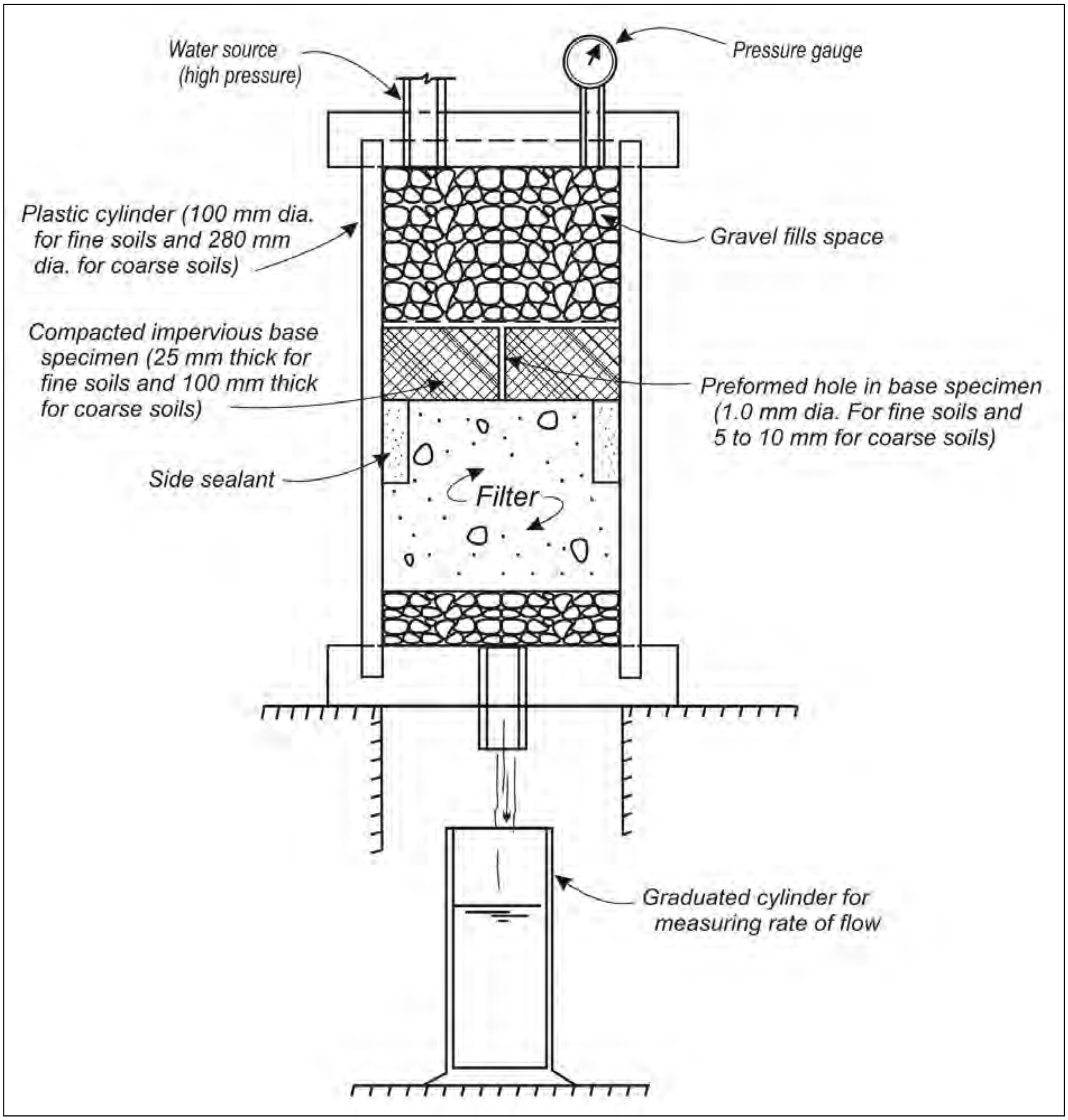

Figure 4-2. NEF Test apparatus.

- *Seal with some erosion-sealing of the concentrated leak but with the potential for some damage and minor to moderate increases in leakage*

- *Partial or no seal with large erosion-slow sealing or no sealing of the concentrated leak, with the potential for large erosion losses, large increases in seepage, and the* *development of sinkholes on the crest and erosion tunnels through the core*

The device used by Foster and Fell to evaluate the potential for continuing erosion is shown in Figure 4-3.

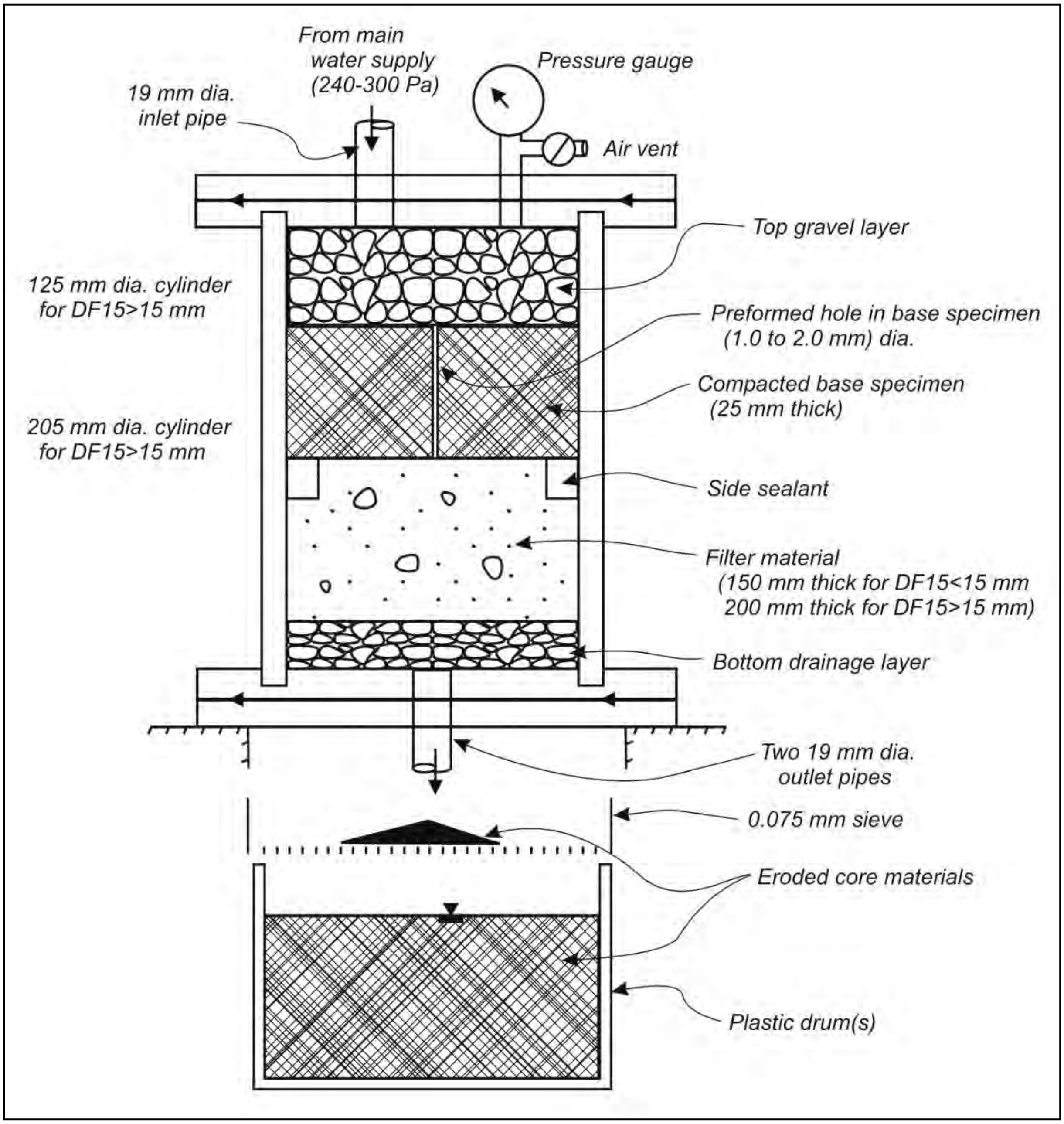

Figure 4-3. CEF test apparatus.

The following modifications were made to the NEF Test during the development of the CEF Test:

- Water passing through the filter during the tests was collected and the eroded materials dried and weighed to determine the loss of base soil required to seal the filter.
- Progressively coarser filters were used until the filter was not sealed.
- Thicker base specimens were used to allow for greater erosion losses.

4.1.2 Rate of erosion tests

Subsequent researchers from the University of New South Wales (Wan and Fell 2004) have used similar experimental setups to study the rates of erosion of soils in which a successful filter is not present. This research focused primarily on the issue of base soil erosion, especially the susceptibility of a given soil to piping. They describe two laboratory tests that were developed to study rates of erosion and the critical hydraulic shear stress necessary to initiate piping erosion. These two tests are (1) the Hole Erosion Test (HET) and (2) the Slot Erosion Test.

The HET uses a 6-mm (0.24-in.) hole drilled in a specimen to model the erosion occurring in an embankment. This contrasts with the 1 mm size of hole used in the NEF Test. The Wan and Fell tests used head differentials of 50 to 1,200 mm (2 in. to 4 ft), whereas the NEF Test used 138 ft of head.

Reclamation became interested in this research since it is useful in risk analysis. Reclamation, as well as other agencies, participated in this research, especially the transition from the hole erosion setup to the slot erosion method (Farrar 2007).

4.1.3 Recommendations

When filter testing is considered necessary for verification of a trial filter gradation design, the procedures by NRCS or Reclamation (Attachment F) should be used.

4.2 Laboratory testing for material quality

This section describes tests that may be used to evaluate the quality of proposed filter materials. Since a critical feature of a filter is to protect against cracks in the base material, it is imperative that the filter itself not sustain a crack. Historically, material quality testing of filters has concentrated on fines content and plasticity of those fines. This was done by using conventional test procedures for gradation analysis (ASTM C117) and plasticity (ASTM D4318). Recent performance has indicated that other types of binders, such as soluble minerals, may also contribute to adhesion in filter materials and that these binding agents may not be detected by conventional test procedures. Therefore, in addition to the conventional test procedures, additional tests are included in this section to more closely evaluate material quality. It is recognized that some of these procedures have not been in general use in the profession and some do not have an accepted standardized test procedure. Tests described in this section are not proposed as a requirement for all filter testing but are described to provide coverage of topics not included in existing guidance.

A particularly good example of the detrimental effect of binding agents can be found in recycled concrete. This material produced by crushing existing concrete, such as paving, is popular for use as a concrete aggregate. Since the gradation range of concrete aggregate is often acceptable as a filter or drain material, it may be attractive to use this in embankment dam construction. However, this material is unacceptable from a quality standpoint since the cement continues to hydrate, even many years after initial placement. This hydration can lead to the material obtaining strength and subsequently sustaining a crack. Therefore, aggregate derived from concrete recycling should never be used for filter or drain material in embankment dams.

Filter and drain materials are derived from clean sands and gravels similar to aggregates (sand and gravel) that are used for production of concrete. It is not surprising then that material quality testing used for aggregates can also be used for filter and drain material. A variety of tests are available to evaluate aggregate quality. It is noted that independent of material testing, qualitative statements have been used in specification paragraphs for both aggregates and filter/drain material. A typical specification statement, as presented by the Federal Highway Administration (2006), is:

> *Aggregates used in concrete mixtures for pavements must be clean, hard, strong, and durable and relatively free of*

absorbed chemicals, coatings of clay, and other fine materials.

While such a statement may inform the contractor of intent, it is difficult to enforce since the requirement is subjective. The test procedures presented in this section are beneficial in specifying the quality requirements for a given material.

Background on material source selection is presented in Attachment A, and a general discussion of material types is presented here {Link_022}. Additional discussion is presented here {Link_023}.

4.2.1 Sampling

An important part of testing aggregates is obtaining a representative sample for the tests to be performed. The U.S. Army Corps of Engineers (USACE) Engineering Manual 1804, Geotechnical Investigations – Appendix F, Chapter F-12, "Sampling from Stockpiles and Bins, Transportation Units, or Conveyor Belts" describes procedures to follow when obtaining samples of aggregates for quality control or quality assurance testing. ASTM D75, Standard Practice for Sampling Aggregates, also describes sampling techniques. American Association of State Highway Transportation Officials (AASHTO) Test Method T2 describes sampling methods for aggregates. In all cases, the sample must be large enough to represent the material accurately. ASTM D75 includes the following minimum sizes of samples of aggregates:

Maximum Size of Aggregate (mm)	Minimum Sample Size (kilograms)	Minimum Sample Size (pounds)
Fine Aggregate		
2.36 mm (No. 8 sieve)	10	22
4.75 mm (No. 4 sieve)	10	22
Coarse Aggregate		
9.5 mm (3/8 in.)	10	22
12.5 mm (1/2 in.)	15	33
19.0 mm (3/4 in.)	25	55
25.0 mm (1 in.)	50	110

37.5 mm (1.5 in.)	75	165
50 mm (2 in.)	100	220

4.2.2 Tests for clay lumps and friable particles

AASHTO Test T112 and ASTM C142 are used to determine the presence and amount of clay lumps and friable particles. Specifications may require a maximum value for acceptance. Samples are soaked 24 hr in distilled water, and any particles that can be broken by finger pressure and removed by wet sieving are classified as clay lumps or friable material. For aggregate acceptability, ASTM C 33 allows no more than 3% clay lumps or friable particles as measured in this test.

4.2.3 Soundness tests

One test for particle soundness is ASTM C 88, Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate. For acceptability, ASTM C 33 limits the average loss during five cycles of the soundness test to 10% when sodium sulfate is used or 15% when magnesium sulfate is used. This requirement should also be met for filters.

Another particle soundness test is ASTM C 131, Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine. For acceptability, ASTM C 33 requires no more than 50% loss during abrasion tests, and this requirement should also be used for filter material.

4.2.4 Tests for plasticity of fines

Silt-size fines are less problematic from a plasticity standpoint than fines that are clay size. For this reason, filter specifications often contain language concerning the plasticity of any fines in the sample. Specifications commonly require that any fines in the filter be nonplastic, as measured in ASTM Standard Test Method D4318. This test for plasticity requires obtaining at least 20 g of material finer than the No. 40 sieve. In cleaner samples, a large amount of filter material may have to be sieved to perform this test. Usually, the only test required is the plastic limit test. To demonstrate that fines are nonplastic, it is only necessary to demonstrate that the sample cannot be rolled out at any water content to a 1/8-in.-diam thread.

4.2.5 Sand equivalent test

The ASTM test procedure for the Sand Equivalent Test is ASTM D2419, and the AASHTO Standard Test Method is T 176. This test is more commonly used in specifying the quality of aggregates used in the manufacture of concrete, but is useful in specifications for filters as well. Historically, the test has not been used as frequently in specifications for filter aggregates.

The Sand Equivalent Test is used to determine the relative proportions of fines or claylike material in fine aggregates. Aggregate passing the 4.75-mm (No. 4) sieve is placed in a graduated, transparent cylinder that is filled with a mixture of water and a flocculating agent. After agitation and 20 min of settling, the sand separates from the flocculated clay, and the heights of sand and clay in the cylinder are measured. The sand equivalent is the ratio of the height of the sand to the height of clay multiplied by 100. A higher sand equivalent value (SEV) indicates cleaner fine aggregate. Minimum specified SEVs for fine aggregate in asphalt mixtures range from 25 to 60. Concrete aggregate specifications commonly require a value to be above 70 or 80. A value greater than 80 is considered by some experts as appropriate for filter material.

4.2.6 Petrographic analysis

A petrographic analysis is another test that is not used frequently for evaluating aggregates proposed for a filter source. The provided information is included here because the testing has occasionally been used for important projects and those where the potential for cementing agents in the aggregate are thought to be a possible problem. ASTM C 295, Standard Guide for Petrographic Examination of Aggregates for Concrete, provides additional documentation of the quality of aggregates used for filters. Factors evaluated in the procedure include:

- Whether the aggregate contains chemically unstable minerals
- Whether the aggregate particles are composed of weathered particles
- Determination of the proportions of cubic, spherical, ellipsoidal, pyramidal, tabular, flat, and elongated particles in an aggregate sample or samples
- Identification of potentially alkali-silica reactive and alkali-carbonate reactive constituents, determination of such constituents

quantitatively, and recommendation of additional tests to confirm or refute the presence in significant amounts of aggregate constituents capable of alkali reaction in concrete
- Identification of contaminants in aggregates, such as synthetic glass, cinders, clinker, coal ash, magnesium oxide, calcium oxide, etc.

These factors are important for material quality in filters since they typically indicate when binding agents may be present. Chemically unstable minerals, or minerals that can go into dissolution, can be re-distributed through the soil mass and coat larger pieces of aggregate, binding them together. A similar process can occur through alkali reaction.

The assessment of particle weathering and particle shape provides an indication of particle strength. Weathered particles will be weaker than particles that have experienced little weathering. Particles exhibiting a more cubic shape are generally stronger than flat, tabular, ellipsoidal, spherical, or elongated shapes.

ASTM Standard Test Method C 294, Standard Descriptive Nomenclature for Constituents of Concrete Aggregates, is also useful in documenting aggregates properties. It includes thorough descriptions of the various rock types commonly used in the production of aggregates.

4.2.7 Vaughan test for cohesion

Vaughan and Soares (1982) introduced a test to evaluate the self-healing properties of a filter zone in an embankment. Their interest in self-healing properties arose from the problems that developed at the Balderhead Dam in England. Vaughan proposed a test (sometimes referred to as the Sand Castle Test) to evaluate the cracking potential of filter material. Vaughan discusses this as follows:

> *For a filter to be effective if cracks form, it is necessary for it to be noncohesive. If it is not, then it may itself sustain an open flooded crack without collapse and so fail to protect a cracked core. The inclusion of more fines in a filter to enable it to retain material of clay floc size may give it cohesion.*

Vaughan goes on to describe a test that he recommended to evaluate this property as:

> *A simple test, suitable for use in a field laboratory, has been devised to examine filter cohesion. It consists of forming a cylindrical or conical sample of moist compacted filter, either in a compaction mould, or in a small bucket such as is used by a child on a beach; standing the sample in a shallow tray (if a bucket is used the operation is exactly as building a child's sand castle) and carefully flooding the tray with water. If the sample then collapses to its true angle of repose as the water rises and destroys the capillary suctions in the filter, then the filter is noncohesive. Samples can be stored for varying periods to see if cohesive bonds form with time. This test is, in effect, a compression test performed at zero effective confining pressure and a very small shear stress, and it is a very sensitive detector of a small degree of cohesion.*

The USACE recommends using the test in their manual on embankment seepage control as:

> *Also, the amount and type of fines present influence the capacity of a filter to self-heal by collapsing any cracks within the filter (see Figure 8-3) [now Figure 4-4]. Therefore, the maximum% fines and type (silt, clay, etc.) to be allowed in the filter of an earth dam must be shown to be sufficiently pervious by laboratory filter tests (I) and self healing by collapse tests (Vaughn 1978).*

Photographic results of successful and unsuccessful material performance based on the USACE procedure are shown in Figures 4-5and 4-6, respectively.

The lack of precision and the inability to express results quantitatively is a shortcoming of this test {Link_024}. Specimen preparation has also been identified as an issue. A more specific preparation procedure is presented in Attachment F, Part V. A curing step is added to the procedure as a more rigorous test of the material. By observation, it has been noted that filter material placement can be exposed to drying and warm summertime temperatures between placements, sometimes for several days. It is thought

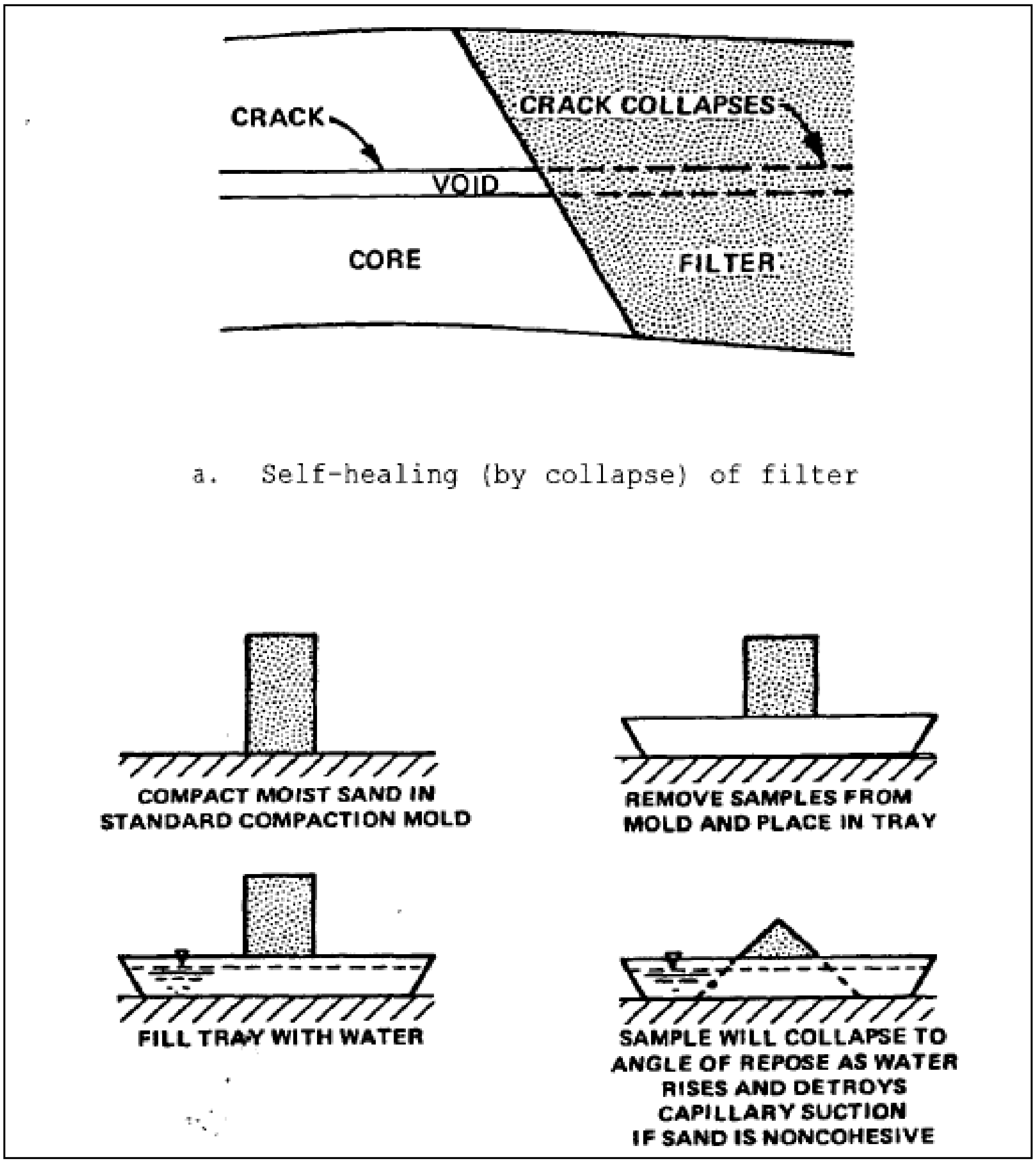

Figure 4-4. Figure 8-3 from USACE Engineering Manual EM 1110-2-1901.
The figure illustrates the Vaughan Test.

that these conditions may contribute to forms of physiochemical bonding between soil grains.

4.2.8 Compressive strength test

Cementing of filters by drying can lead to a filter sustaining a crack rather than preventing one. For this reason, tests to detect cementitious properties of filters should be considered in environments where filters may be

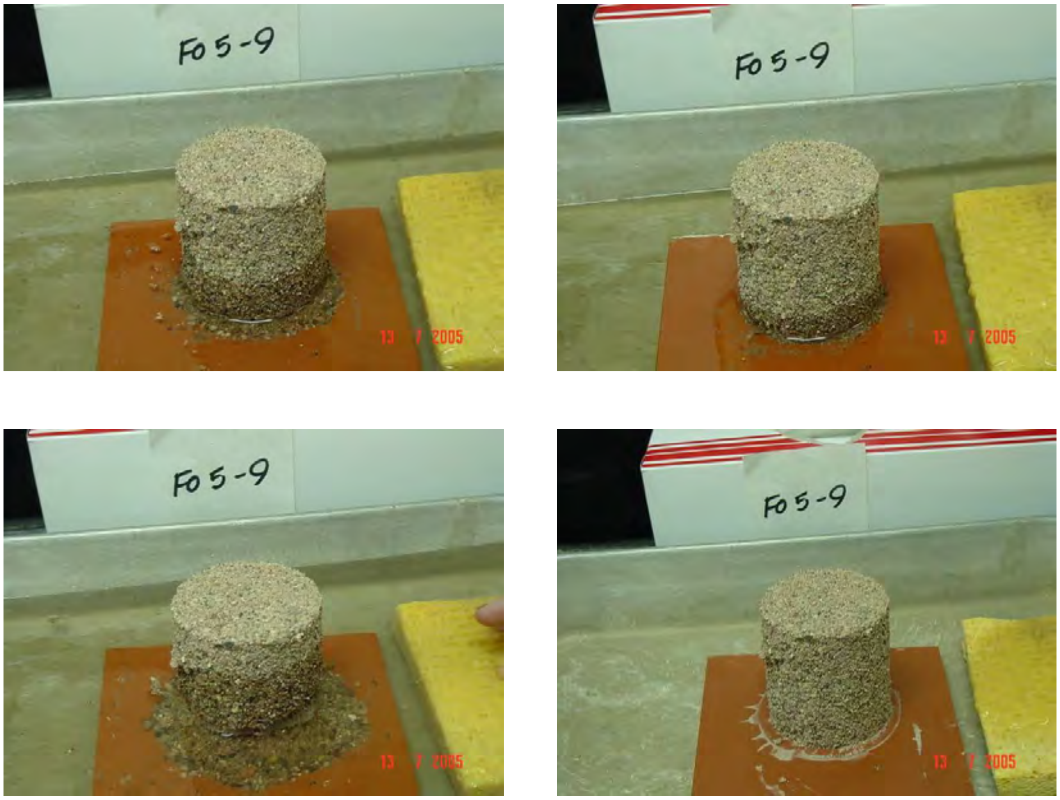

Figure 4-5. Illustration of relatively poor self-healing behavior. The sample does not collapse well after 50% submersion. The test sequence begins at the lower right photo and progresses counter-clockwise, ending in the lower left photo.

subjected to drying after placement. An example of such a test was proposed by McCook (2002). The paper has the following discussion:

> *Compressive strength tests on filter sands may be helpful in identifying sands with cementitious properties. These tests should be considered for testing and qualifying sands proposed for filters and drains in important projects. The low cost and ease of performing the test with already available equipment are factors encouraging the wider use of the test. Additional research is needed to explore how important factors such as molding water content affect the results. Additional research is also needed to establish a value of compressive strength that is excessive. Preliminary results show that filters commonly have compressive strengths less than 20 psi, but whether that value is an appropriate maximum allowable value is not clear with available data. An*

Figure 4-6. Illustration of relatively good self-healing behavior. The sample collapses relatively quickly as it is submersed.

appropriate allowable level could be lower, perhaps 10 psi. A level that is too restrictive might eliminate locally available aggregates and substantially increase the cost of hauling in aggregates from a more distant source. Some means of tying the results of this test to field performance is needed.

The procedure for measuring the compressive strength is included in Attachment F.

4.2.9 Summary of test procedures for determination of material quality

The following table summarizes tests normally performed on aggregates and are applicable for filter materials:

Test	ASTM Standard Test Method	AASHTO Standard Test Method
Gradation (sieve analysis)	C-136	T-27
Gradation (fines)	C-117	T-11
Specific gravity (coarse aggregates)	C-127	T-85
Specific gravity (fine aggregates)	C-128	T-84
Clay lumps and friable particles	C-142	T-112
Soundness (sodium or magnesium sulfate)	C-88	T-104
Soundness (Los Angeles abrasion test)	C-131	T-96
Sand equivalent	D2419	T-176
Petrographic examination	C-295	

The use of these tests is dependent on prior knowledge of available source material and judgment. If it is suspected that the source aggregate is of questionable quality, several of these tests can be used. If the source material has been successfully used previously or the aggregate is judged to be of high quality by visual examination, then fewer tests procedures would be utilized.

It should also be recognized that many commercial pits will have the results of several of these procedures and that data can be used in assessing the source. If the data are not available from the supplier and the quality of the aggregate is in question, material quality testing described in this section should be performed. Additionally, any test procedure that is required in the specification should be performed prior to solicitation to assure the source is acceptable.

4.2.10 Recommendations

- Plasticity of fines of filter material should always be measured using ASTM D 4318.
- When the quality of the candidate material is suspect, the self-healing potential should be measured using the modified Vaughan or Compressive Strength Tests.
- Quality requirements for filter material should be specified using test results and not qualitative statements.
- Due to material quality issues, aggregate obtained from concrete recycling operations should never be used for filter or drain material in embankment dams.

5 Filter Design Procedure

5.1 Background

The base soil is the core (designed water barrier) material whose integrity must remain uncompromised during the dam's life cycle. The filter soil acts as the protective device or "fail-safe" mechanism to ensure proper functioning of the core material. The filter soil particles are coarser-grained than the base soil particles, to achieve the purposes discussed in greater detail elsewhere in this manual.

This chapter presents a step-by-step procedure for selecting the proper gradation band of a filter or drainage material whose purpose is to protect a base soil material. The procedure applies to zones used in embankment dams, foundation seepage collection zones such as toe drains, or any other application where seepage occurs and particle movement is to be prevented. This procedure can be used in both single- and multi-stage filter applications. For multistage applications, the procedure is repeated for each zone boundary progressing from the finest to the coarsest grained soils.

Filter gradation limits achieved by this procedure will be a balance between permeability requirements on the finer-grained particle distribution side and particle retention requirements on the coarser-grained particle distribution side. The window of fine-to-coarse limits allows for flexibility in selection of the optimum filter gradation band required to achieve the intended goal of the filter.

The procedure is primarily based on research performed at the Natural Resource Conservation Service (NRCS) in the 1980s (Sherard 1984). That research also influenced procedures used by NRCS, U.S. Army Corps of Engineers (USACE), and Bureau of Reclamation (Reclamation). While most design criteria are based on historical research by Sherard and others, there are some differences between the procedures of each of these U.S. Government agencies. These are elaborated on in {Link_015}. Additional research performed in the past decade by Foster and Fell (2001) and others has contributed to the awareness of dispersive clay base soils and how they should be filtered. The procedure included in this chapter is a compilation of the information from these sources.

5.1.1 Selection of base soil gradation

The first step in the filter design procedure is to determine the representative gradation of the soil being protected. Historically, design guidance has indicated a single gradation with little explanation of how that gradation is obtained. USACE {Link_016} and Reclamation {Link_017} provide narrative assessments on base soil selection, and detailed considerations are addressed in Attachment A. The information presented in Attachment A is intended to elucidate which factors should be considered when evaluating base soil data and choosing a representative gradation. The information should be used as a guide rather than strict procedural requirements for base soil selection.

Base soil selection is complicated by soil variability as it is represented in gradation tests. Variability will be less for embankment fill since there is blending and mixing of the source material as it is excavated from the borrow area and placed in the dam. On the other hand, foundation material will have a greater degree of variability and present a greater challenge in base soil selection. Foundation soils also present a challenge in that the selection of accurate base soil gradations is only as good as the understanding of the geology. If the lithology of the subsurface deposits is poorly understood, this can lead to incorrectly grouping multiple soil gradations, resulting in a too coarse or too fine a filter for a given geologic unit. Probably the most difficult geologic conditions to quantify are undifferentiated units. These are soil deposits that usually have limited areal extent and do not warrant mapping them as unique soil layers. This may result in a broad range of soil types for consideration during base soil candidate selection.

Consideration should also be given to sampling errors, classification errors, and so-called outliers. Invariably, when numerous samples are collected and obtained in earth materials, there will be one or two samples that do not appear to match all others, even when the sampled layer is thought to be homogenous. This variation can come from variability of the materials themselves or from collection or laboratory (testing) errors. When an outlier is on the finer side of the candidate gradations, a problem can arise if it is used as the representative base soil gradation since it will result in a too-fine filter being designed.

Since foundation soils typically have greater variability than earthfill materials, as described above, the base soil selection procedure is different for these two classes. As would be expected, the more variable class has a longer list of characteristics that needs to be evaluated (see Figure A-15), and the less variable material is simpler (see Figure A-14).

5.1.2 Dispersive clay base soil considerations

For base soils with more than 15% fines, adequate tests should be performed to establish whether the clay fines are dispersive in character. The crumb test (ASTM D 6572) and double hydrometer test (ASTM D 4221) usually define this property adequately, but in some cases, pinhole, ASTM D 4647, and chemical tests may also be required. The NRCS reference, "Chapter 13, Part 633 of the National Engineering Manual, Dispersive Clays," contains useful advice for sampling and testing for dispersive clays as does the Reclamation reference, "R-91-09, Characteristics and Problems of Dispersive Clay Soils."

As the name implies, dispersive clay minerals tend to "come apart" when immersed in fresh water, as opposed to flocculation (come together), which is seen in all other types of clays. This dispersion tends to make the nominal particle sizes effectively smaller than what is measured in non-dispersive samples. Since the effective particle sizes are smaller, the retention rules based on a D15 size are not entirely representative. A different set of retention criteria are used, as described later in this chapter.

5.1.3 Base soil computational re-grading

Computationally re-grading the base soil (i.e., calculating on paper instead of field sorting) at the beginning of the filter design procedure is a critical step that must be followed, when applicable, in order to obtain a correctly designed filter. The concept of computational re-grading was developed by Sherard to correct for broadly graded soils. Broadly graded base soils can be internally unstable (i.e., inadequate particle retention), and re-grading corrects for this phenomena. Permitting the inclusion of gravel (> sieve No. 4 size) within a base soil gradation will lead to a large D85B size and subsequently a large D15F size. Since gravel particles do not have any particle retention capability in broadly graded or gap-graded soils, the resulting filter gradation will be too coarse to provide particle retention of the finer fraction of the base soil (i.e., the filter will not meet particle retention criteria for the base soil). The exception to this rule is that soils with less than 15% fines do not require re-grading.

The procedure for base soil computational re-grading is illustrated in Figure 5-1, with the steps listed below:

Sieve Size	Original Percent Passing	Adjustment	Final Percent Passing (Re-graded gradation):
3"	100.0		
1 1/2"	85.7		
3/4"	74.6		
3/8"	65.9		
#4	57.9	(57.9 / 57.9) x 100	100.0
#8	54.6	(54.6 / 57.9) x 100	94.3
#16	49.0	(49.0 / 57.9) x 100	84.6
#30	42.6	Typ. (42.6 / 57.9) x 100	73.6
#50	32.2	(32.2 / 57.9) x 100	55.6
#100	19.8	(19.8 / 57.9) x 100	34.2
#200	13.0	(13.0 / 57.9) x 100	22.5
1 min	9.9	(9.9 / 57.9) x 100	17.1
4 min	5.4	(5.4 / 57.9) x 100	9.3
19 min	2.9	(2.9 / 57.9) x 100	5.0
60 min	1.6	(1.6 / 57.9) x 100	2.8

Figure 5-1. Example showing computational re-grading to the No. 4 sieve size.

1. Obtain a correction factor (or adjustment ratio) by dividing 100 by the percent passing the No. 4 (4.75-mm) sieve size of the base soil.
2. Multiply the percentage passing each sieve size of the original base soil by the correction factor (or adjustment ratio).
3. Plot these adjusted percentages to obtain the computationally re-graded gradation curve.
4. Use the re-graded curve plot to determine the percentage passing the No. 200 (0.075-mm) sieve to use in step 4 below.

The problem of not re-grading the base soil gradation is illustrated graphically in Figures 5-2 and 5-3. Figure 5-2 shows a base soil that has not been re-graded (i.e., original base gradation curve is shown). Sizing a filter for this material results in a filter consisting primarily of coarse gravel, as

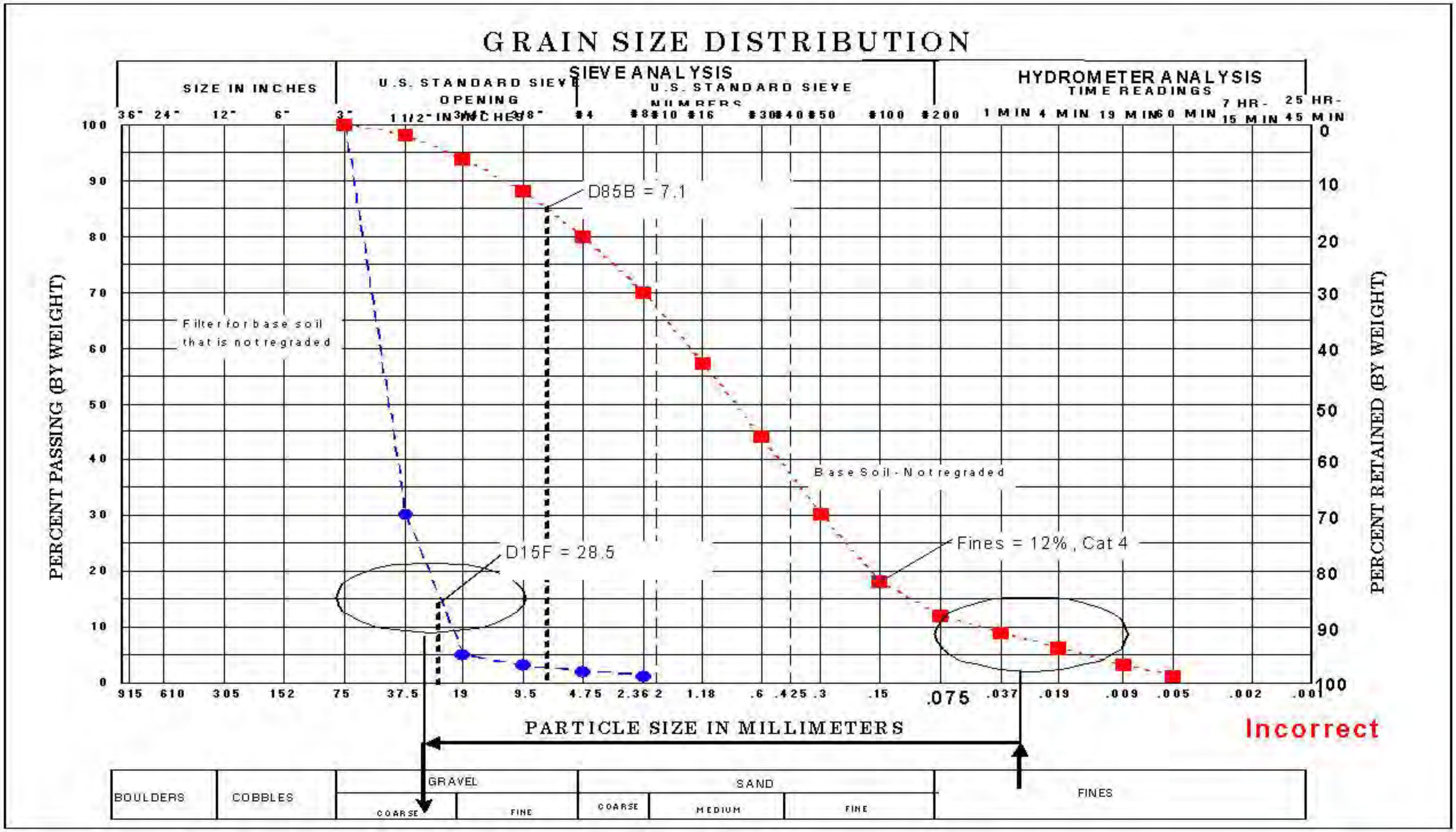

Figure 5-2. Illustration of an incorrectly designed filter gradation (blue line) because the base soil gradation (red line) was not computationally re-graded to the No. 4 sieve size.

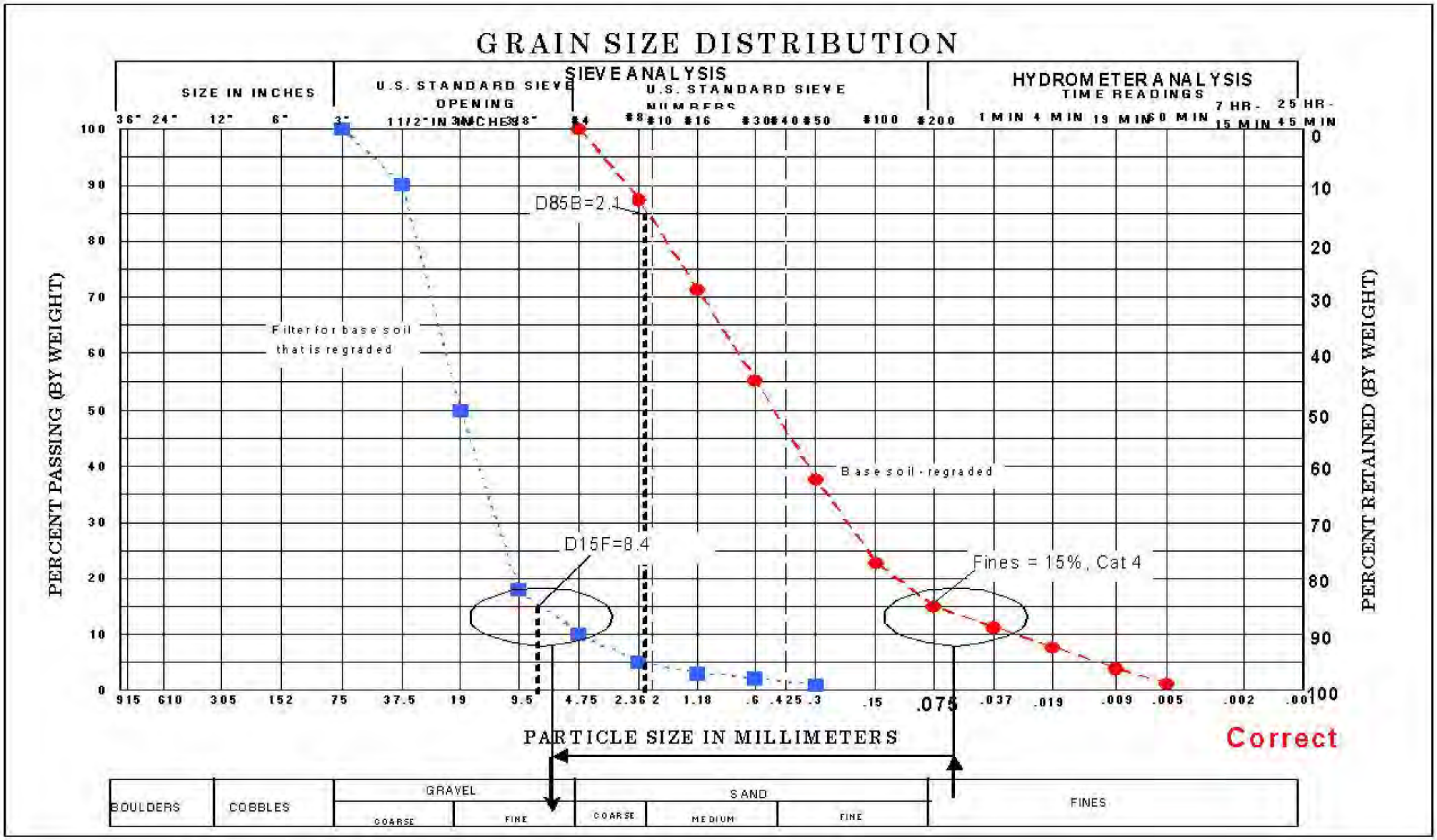

Figure 5-3. Illustration of the original base soil material as shown in Figure 5-2 after computational re-grading (red line). Re-grading results in a correctly sized (slightly finer-grained) filter (blue line).

shown on the figure. This design results in the silt and fine sand of the base material eroding through the voids in the coarse gravel filter.

Figure 5-3 shows the same base soil computationally re-graded beginning with the No. 4 sieve size. The filter design based on the re-graded soil is a fine gravel with 10% sand. This design will not permit movement of the silt and fine sand of the base soil through the sand and fine gravel filter.

5.2 Filter design procedure

The following section provides a step-by-step procedure based on Sherard's research, guidance of Federal agencies, and other studies in the last decade. More detailed discussions are found in Attachments D, G, and {Link_027}.

Step 1: Plot the gradation curve(s) (grain-size distribution) of the base soil material(s). Determine if the base soils have dispersive clay content and note it for later use in the procedure.

Step 2: Determine if the base soil(s) have particles larger than the No. 4 sieve (i.e., gravel sizes). Also, determine if the base soil(s) are gap-graded, thus potentially subject to internal instability (reduced particle retention capability).

(a) If the base soil has no gravel particles, proceed to Step 4.

(b) If a base soil contains any particles larger than the No. 4 sieve, the soil should be computationally re-graded on the No. 4 sieve (go to Step 3), with the following exception: sands and gravels with less than 15% fines that are not gap-graded and not broadly graded do not require re-grading (proceed to Step 4).

A flowchart illustrating the Step 2 process is shown in Figure 5-4.

If the base soil is gap-graded (i.e., missing medium grain sizes), the coarse grains may not deter the migration of the finer grains. The filter should be designed to protect the finer grains rather than the total range of particle sizes. USACE EM 1110-2-2300 (30 July 2004) illustrates how a gap-graded base soil may be re-graded on the No. 30 sieve (identical in fashion to the above procedure for re-grading on the No. 4 sieve), and the filter

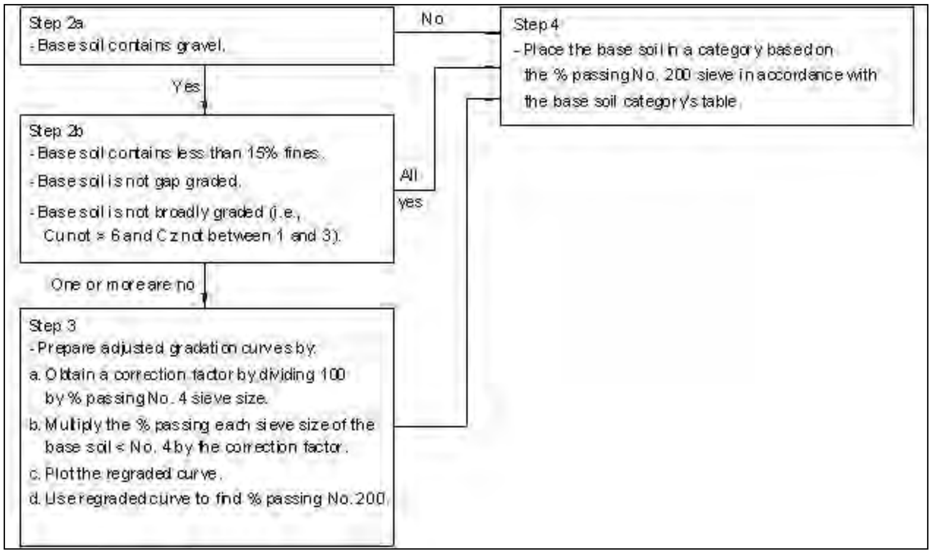

Figure 5-4. Flowchart of the Step 2 process.

design is based on the re-graded curve. The resultant filter design should be checked with filter testing to verify its performance.

Step 3: Prepare adjusted re-graded gradation curves (i.e., re-graded) for base soils that have particles larger than the No. 4 (4.75-millimeter [mm]) sieve.

Refer to either previous Section 5.1.3 or the above illustration for the re-grading procedure.

Step 4: Determine base soil category based on percent passing the No. 200 (0.075-mm) sieve in accordance with the following table.

Table 5-1. Base soil categories.

Base Soil Category	Percent Finer Than No. 200 Sieve (0.075-mm) (after re-grading where applicable)	Base Soil Description
1	> 85	Fine silt and clays
2	40 – 85	Sands, silts, clays, and silty and sands
3	15 – 39	Silty and clayey sands and gravels
4	< 15	Sands and gravels
Note: Table 5-1 is the same for USACE, Reclamation, and NRCS guidance (Table 2, USBR Design Standards No. 13(5); Table B-1, EM 1110-2-2300; Table D-1, EM 1110-2-1901; and NRCS Table 26-1)		

Step 5: To satisfy particle retention (internal stability) requirements, calculate the maximum allowable $D_{15}F$ size in accordance with the following table. Selection is based on the $D_{85}B$ of the re-graded (if applicable) base soil. Plot the result (maximum allowable $D_{15}F$ size) as a single point on a preliminary design plot (illustrated in Section 5.3).

Table 5-2. Filtering criteria.

Base Soil Category	Filtering – Maximum $D_{15}F$
1	The maximum $D_{15}F$ should be ≤ 9 × $D_{85}B$, but not less than 0.2 mm, unless the soils are dispersive. Dispersive soils require a maximum $D_{15}F$ that is ≤ 6.5 × $D_{85}B$ size, but not less than 0.2 mm.
2	The maximum $D_{15}F$ should be ≤ 0.7 mm unless soil is dispersive, in which case the maximum $D_{15}F$ should be < 0.5 mm.
3	For nondispersive soils, the maximum $D_{15}F$ should be $\leq \left[\frac{40-A}{25}\right]\left[(4\times D_{85}B)-0.7mm^*\right] + 0.7mm^*$ where: A =% passing No. 200 sieve after any re-grading. When 4 × $D_{85}B$ is less than 0.7 mm*, use 0.7 mm* * - For dispersive soils, use 0.5 mm instead of 0.7 mm.
4	The maximum $D_{15}F$ should be ≤ 4 × $D_{85}B$ of base soil after re-grading
Note: Table 5-2 has essentially the same criteria as seen in USACE, Reclamation, and NRCS guidance (Table 2, USBR Design Standards No. 13(5); Table B-2, EM 1110-2-2300; Table D-2, EM 1110-2-1901; and NRCS Table 26-2). NRCS adds dispersive soil criteria, and USACE adds wave/surge criteria.	

Step 6: To satisfy permeability requirements, determine the minimum allowable $D_{15}F$:

Minimum $D_{15}F$ ≥ 5 × maximum $D_{15}B$ (Reclamation)
Minimum $D_{15}F$ ≥ 3 to 5 × maximum $D_{15}B$ (USACE)
Minimum $D_{15}F$ ≥ 4 to 5 × maximum $D_{15}B$ (NRCS)
Minimum $D_{15}F$ is computed prior to any re-grading, if any, and should not be smaller than 0.1 mm.

Plot the result (minimum allowable $D_{15}F$ size) as a single point on the preliminary design plot (illustrated in Section 5.3).

Step 7: Limit the width of the filter band and prevent gap-graded filter design. After plotting the maximum and minimum $D_{15}F$ sizes on the preliminary design gradation plot, check that their ratio is less than or equal to 5 (i.e., maximum $D_{15}F < 5 \times$ minimum $D_{15}F$). In addition, check the D_{10} and D_{60} size limits to ensure coefficient of uniformity (C_u) between 2 and 6.

Plot the results as points on the preliminary design plot (illustrated in Section 5.3).

Additional discussion on preventing gap-graded filters is presented here {Link_019}.

Step 8: To limit the amount of fines and oversized material, determine the minimum D_5F and maximum $D_{100}F$ according to the following table:

Table 5-3. Maximum and minimum particle size criteria.

Base Soil Category	Maximum $D_{100}F$	Minimum D_5F
ALL categories	≤ 2 in. (51 mm)	0.075 mm (No. 200 sieve)

USACE sets maximum size at 3 in. (75 mm), maximum 5% fines passing the No. 200 sieve, and PI equal to zero.

Step 9: To limit segregation potential, determine maximum $D_{90}F$ from the following table:

Table 5-4. Segregation criteria.

Base Soil Category	If Minimum DF is: (mm)	Then, Maximum $D_{90}F$ is: (mm)
ALL categories	< 0.5 0.5–1.0 1.0–2.0 2.0–5.0 5.0–10 10 - 50	20 25 30 40 50 60

Additional discussion of segregation is presented here {Link_019}.

Step 10: Determine the filter gradation band within the control points.

Select a gradation band within the control points (limits). Two methods are presented based on the practice of NRCS and Reclamation.

The NRCS method is:

To prevent use of gap-graded filters, the width of the filter band is adjusted such that the ratio of the maximum diameter at any passing less than 60% is 5 or less. To check this at the $D_{15}F$, divide the maximum $D_{15}F$ by 5, and use the coarsest of the new point. At the D_{60} limits (Step 7 above), the band width can be laterally adjusted to meet the Step 7 requirements. The adjustable band width may be set to accommodate commercially available gradations or other materials available at or near the project site.

The Reclamation method considers the purpose of the filter and provides guidance for those cases. This method, along with examples, is presented in Attachment G.

5.2.1 Drainpipe perforations

If the envelope filter will be used adjacent to a perforated pipe, then:

> *The maximum pipe perforation dimension should be no larger than the finer side of the $D_{50}E$ where $D_{50}E$ is taken from the gradation of the envelope (drain) material that surrounds the drainpipe.*

5.3 Design examples

5.3.1 General example

For the purpose of illustrating the procedures listed above for a single-stage filter design, a hypothetical re-graded base soil curve is shown in Figure 5-5. Steps 1 through 3 are not repeated since the base soil is already computationally re-graded. The purpose herein is not to select the optimum filter to protect this particular base soil, but to illustrate the steps required to accomplish the filter design process.

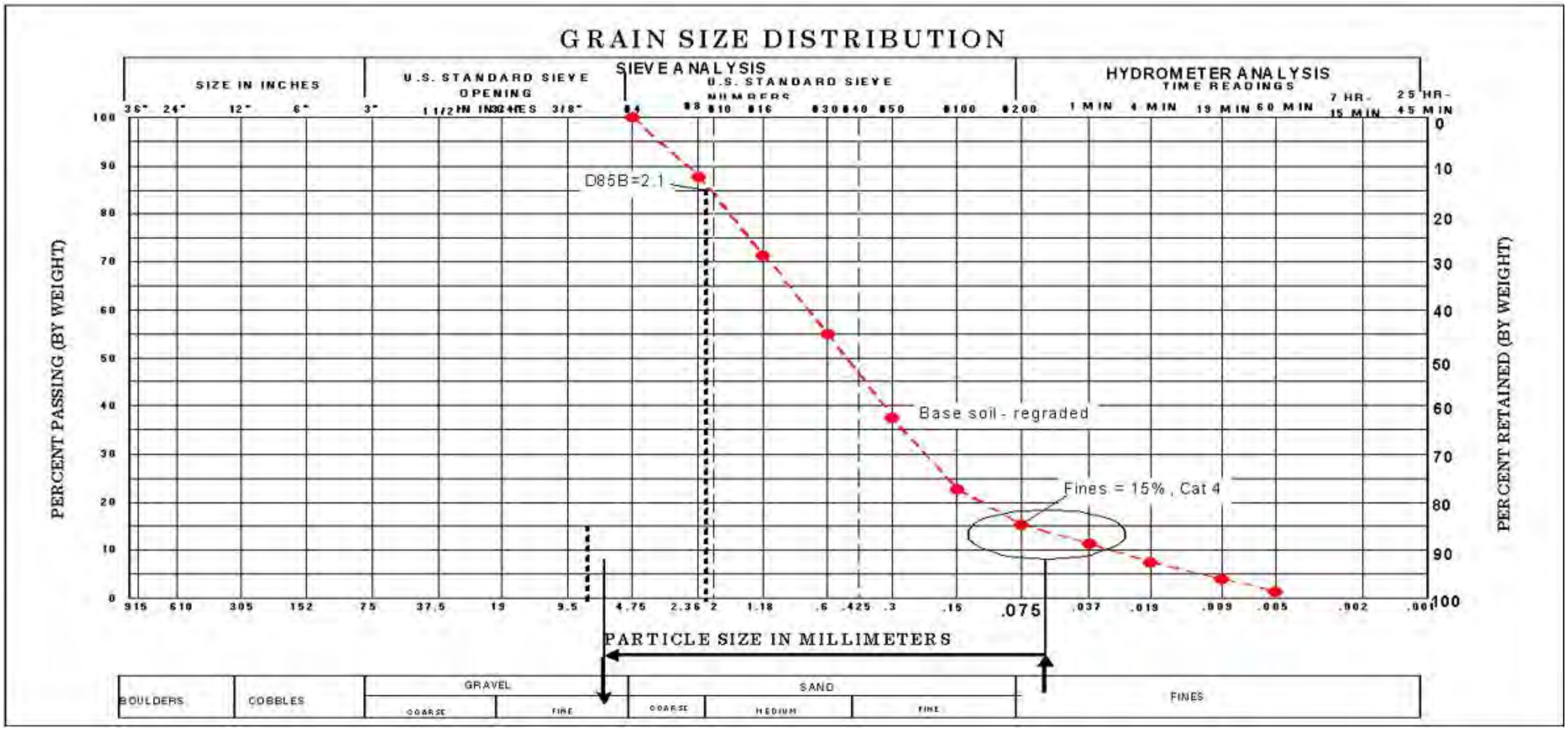

Figure 5-5. Illustrative re-graded base soil curve.

Since the percent fines passing the No. 200 sieve size is 15%, per Table 5-1 this re-graded base soil is in Base Soil Category 4, satisfying Step 4 in Section 5.2.

Per Table 5-2, for a Category 4 base soil, the maximum $D_{15}F$ should be less than or equal to $4 \times D_{85}B$, or $(4 \times 2.1) = 8.4$ mm. Plot the max $D_{15}F =$ 8.4 mm as point A on Figure 5-6 to complete Step 5.

To satisfy permeability requirements (Step 6), calculate the minimum $D_{15}F$ as 1/5th of the maximum $D_{15}F$, or $(1/5) \times 8.4 = 1.68$ m (must not be less than 0.1 mm). Plot the min $D_{15}F = 1.68$ m as point B on Figure 5-6.

An additional check of filter permeability is performed by computing the ratio of the minimum $D_{15}F$ to the $D_{15}B$ of the coarsest soil in the base soil band (before re-grading the soils). A rule of thumb commonly used is that this ratio should be in the range of 4 to 5. Because permeability is proportional to the square of the D_{15} size, filters designed using this guideline will be at least 16 times and up to 25 times as permeable as the base soils with which they are in contact. The minimum allowable $D_{15}F$ may need to be modified in consideration of this additional check. This modification will result in a narrower filter band that may be more difficult to supply, but the increased permeability may justify this more restrictive design. Per Step 7, limit the width of the filter band and prevent gap-graded design. After plotting the maximum and minimum $D_{15}F$ sizes on the preliminary design gradation plot, check that their ratio is less than or equal to 5 (i.e., maximum $D_{15}F < 5 \times$ minimum $D_{15}F$). In addition, check the D_{10} and D_{60} size limits to ensure coefficient of uniformity (C_u) between 2 and 6:

(a) Find the maximum D_{10} size and plot as point C:

C = point A × 0.7 = 8.4 × 0.7 = 5.88 mm

(b) Find the minimum D_{10} size and plot as point D:

D = point B × 0.7 (but not less than 0.075 mm) = 1.68 × 0.7 = 1.17 mm

(c) Find the maximum D_{60} size for $C_u = 6$ (point E):

E = point C × 6 = 5.9 × 6 = 35.4 mm

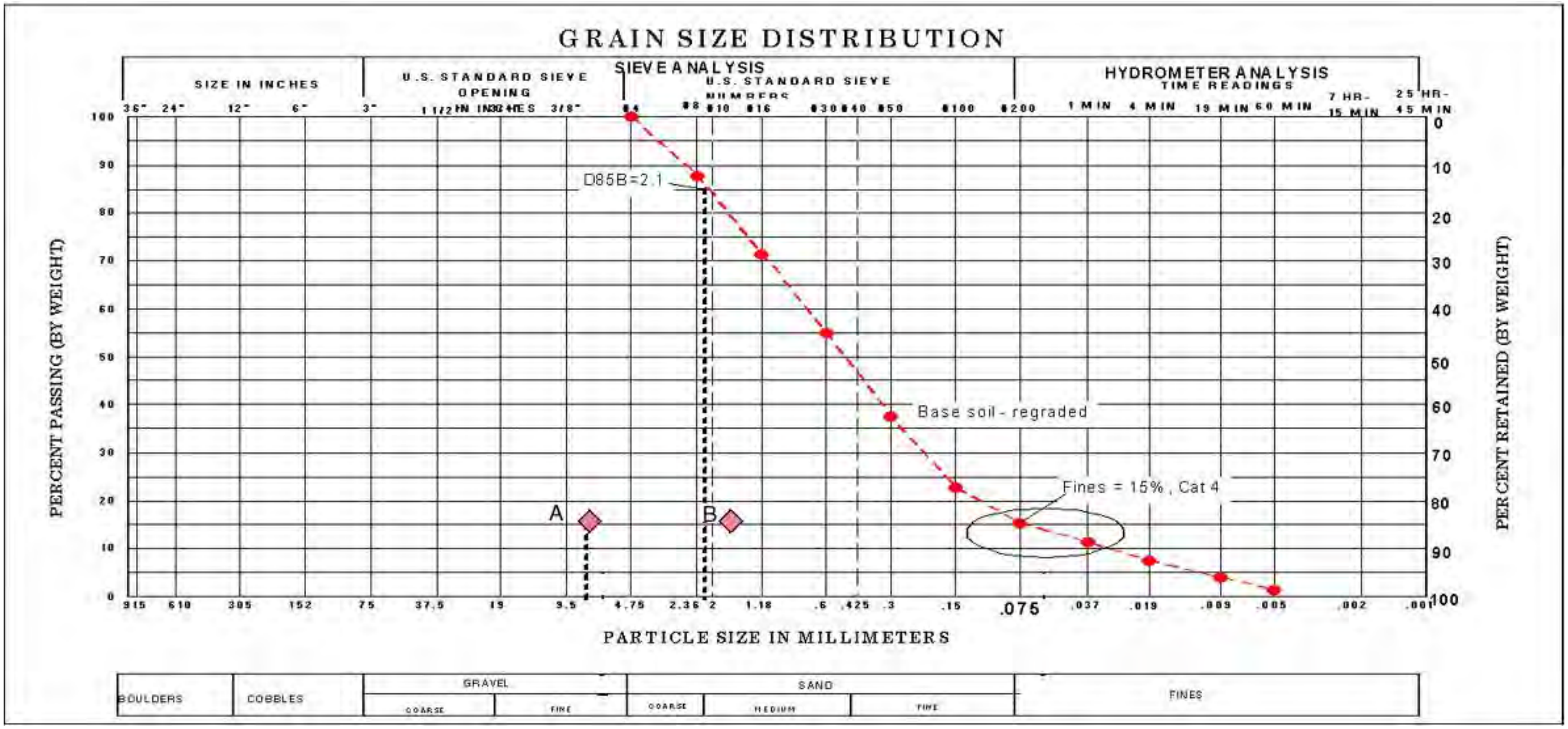

Figure 5-6. Initial control points (A and B) for designing the filter.

(d) Find the minimum D_{60} size for C_u =2 (point F):

F = point D × 2 = 1.17 × 2 = 2.34 mm

(e) Find the size of a "sliding bar" defined by points G and H where:

F < G < E
H = G × 5

As described later, this bar can be adjusted back and forth between points E and F.

An alternative method to controlling the width of the mid-portion of the gradation band using a vertical-sliding bar is described in Attachment D.

Figure 5-7 shows the layout of these points.

Per Step 8, the minimum D_5F size is 0.075 mm (No. 200 sieve), shown as point I on Figure 5-8. The maximum $D_{100}F$ size is < 2 in. (51 mm), shown as point J on Figure 5-8.

Per Step 9, the filter must not be overly broad in order to prevent possible segregation during construction. Since the minimum $D_{10}F$ size is 1.17 mm (from Step 7), the maximum $D_{90}F$ size is 30 mm (from Table 5-4). The maximum $D_{90}F$ size is represented by point K on Figure 5-8.

The final step is to select a preliminary filter gradation band within the control points (limits). Numerous filter material selections are possible, depending on desired optimal filter function (particle retention and/or drainage), commercial availability, and other concerns noted in this manual (see Chapter 6). For example, the NRCS recommendation for preventing selection of gap-graded filters is to adjust the width of the preliminary filter band such that the ratio of the maximum diameter at any percent passing less than 60% is 5 or less. To check this at the $D_{15}F$, divide the maximum $D_{15}F$ (point A) by 5, and use the coarsest of the new point or point B. At the D_{60} limits, the band width determined by points G and H can slide back and forth between points E and F. This sliding band width is set to accommodate commercially available gradations or other

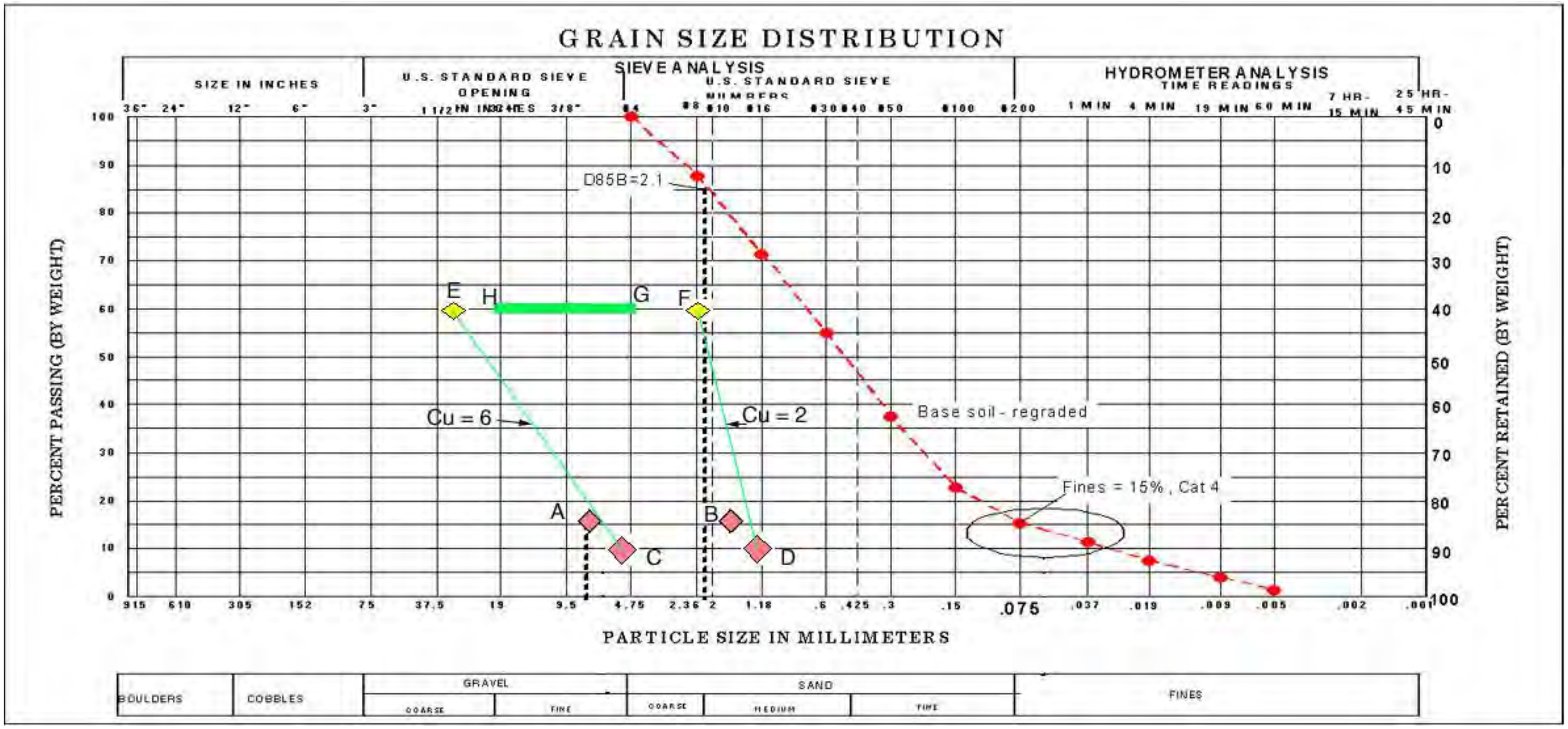

Figure 5-7. Additional control points (C through H) for designing the filter.

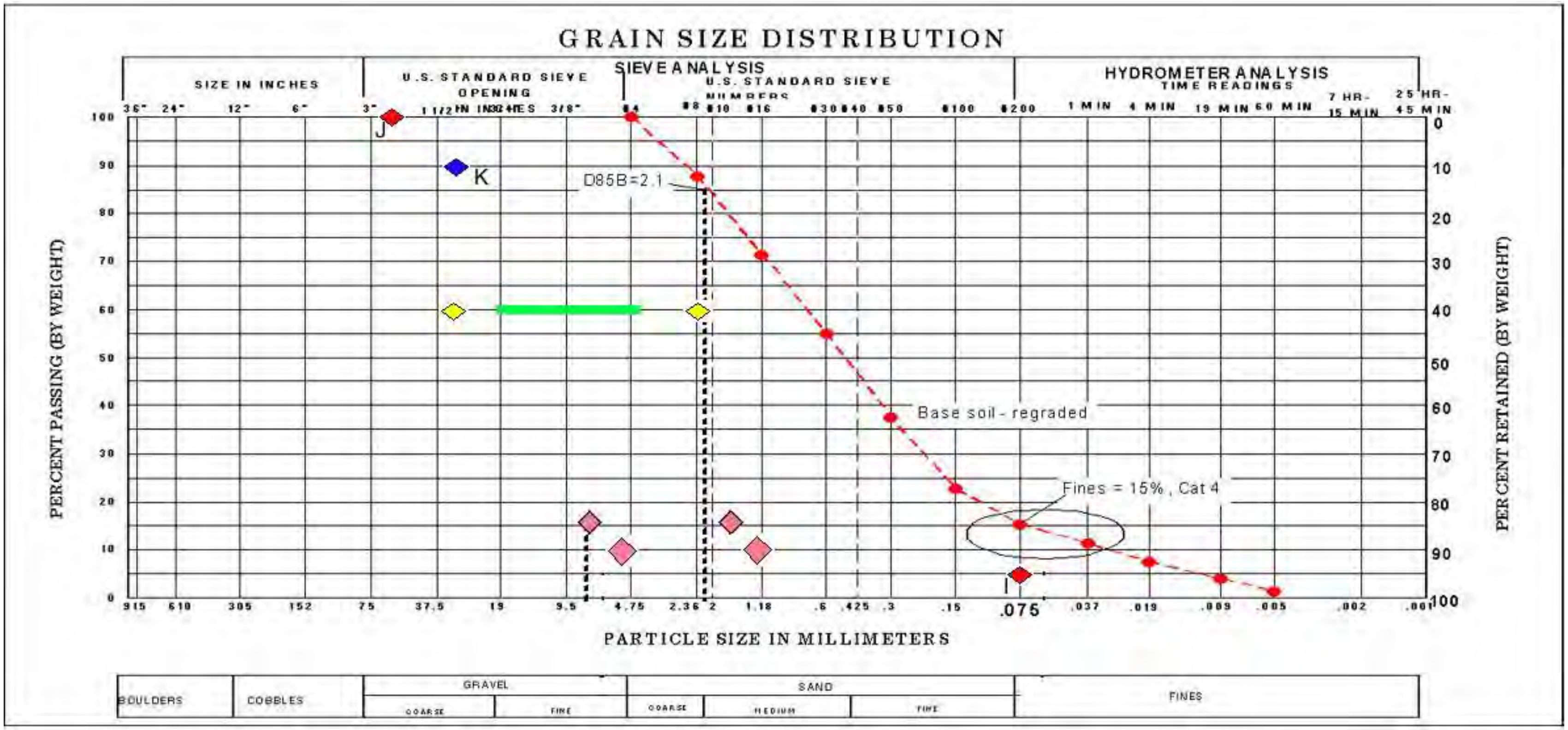

Figure 5-8. Additional control points (I, J, and K) for designing the filter.

materials available at or near the project site. Attachment G lists the Reclamation methods and concerns for proper filter selection.

Figure 5-9 illustrates one possible filter material candidate that fulfills the filter design requirements in the above example. This filter material would have to be evaluated further to examine (1) if it is commercially or readily available, (2) if it matches the designer's goals for a single-stage filter, and (3) if additional design considerations are needed (Chapter 6).

Resources are available for conducting a filter design and iterating to find the most suitable filter material based on commercially-available aggregate mixes. For example, there are numerous standardized soil mixes such as the ASTM C-33 concrete sand that are commercially available and readily supplied by commercial firms. Iterations to match the available (or most economical) filter materials to the required design parameters will be necessary, and computer applications may be enlisted to perform such an endeavor. Navin et al. (2006) provides a comprehensive listing of commercially-available mixes matched to filter design gradations in a user-friendly spreadsheet application.

5.3.2 Detailed example

In this example, a filter is required in the construction of a flood-protection parapet wall along the top of an existing dam as shown on Figure 5-10. The location of the filter material is such that the filter is not associated with a drainage feature, but is functioning as a separation layer between the existing embankment dam core material and the aggregate base course[1] for asphalt paving on top of the dam in an area of potentially elevated seepage gradients directly behind the parapet wall during flood surcharge. The filter (Zone 5) is to protect against piping failure caused by seepage flow under the wall during flood surcharge. Interface 1 is the boundary between the embankment dam core and the filter. Interface 2 is the contact between the filter and the aggregate base course.

The following steps outline the procedure for specifying a filter material for this example. This example checks for filter compatibility at the two

[1] "Aggregate base course" is the standard naming convention for a pavement sub-base. This "base" should not be confused with the base soil used elsewhere in this example.

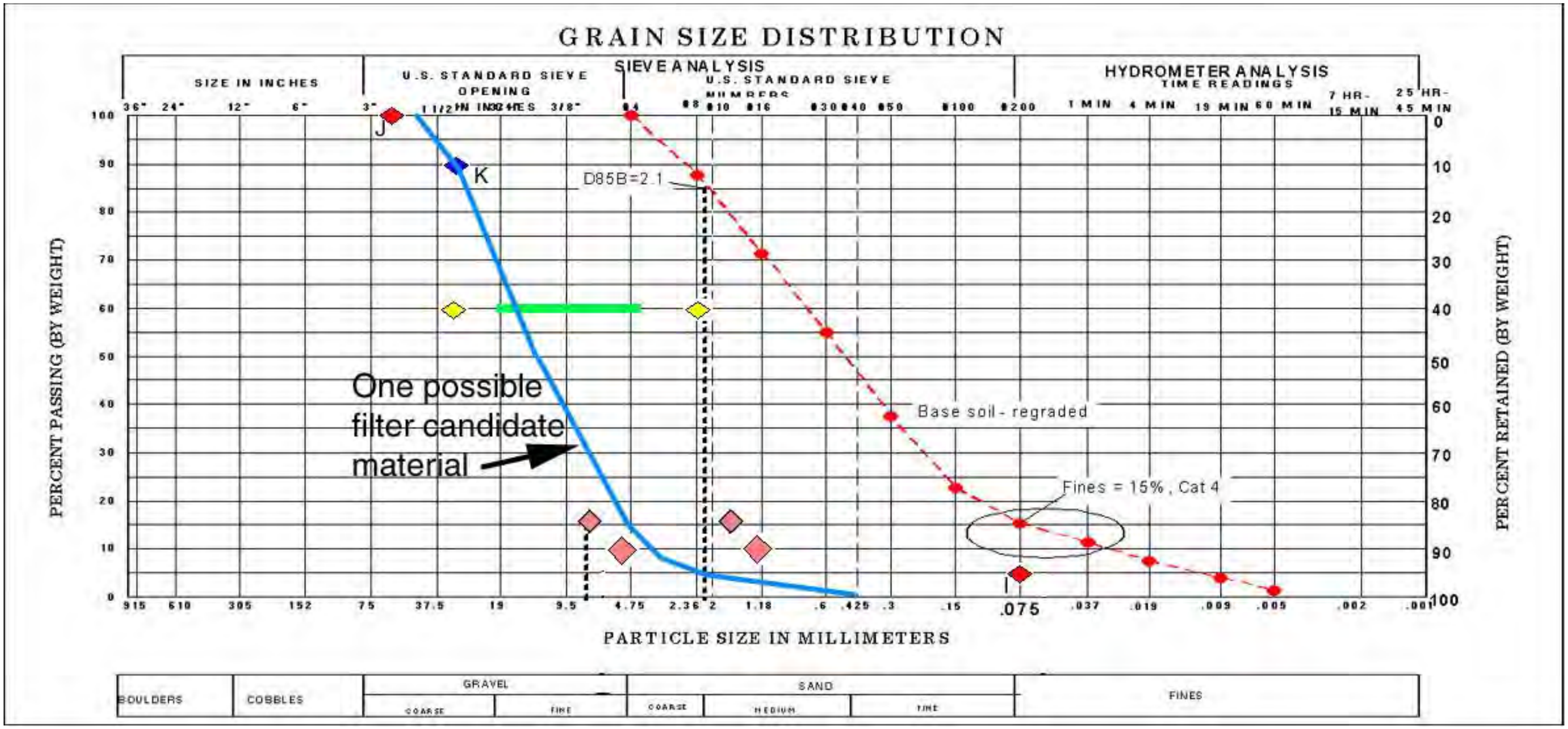

Figure 5-9. The filter design process is completed when a candidate material is evaluated and selected to function as an optimum first-stage filter.

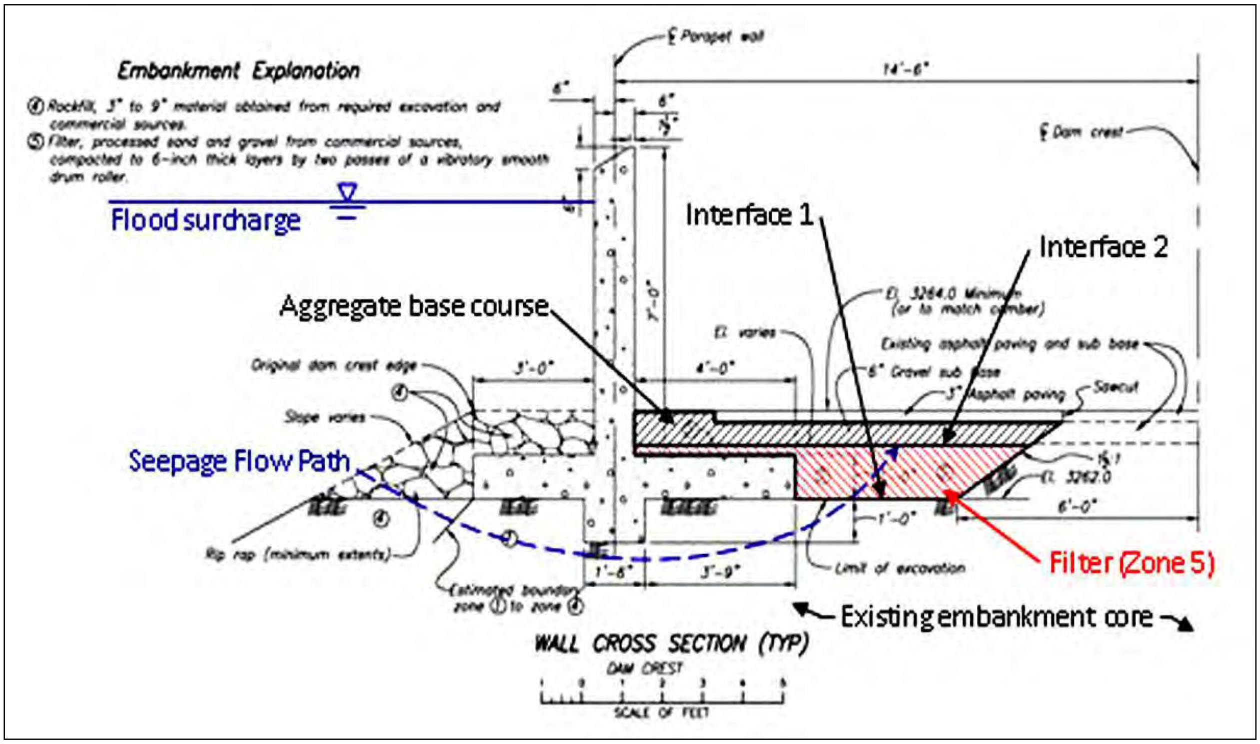

Figure 5-10. Parapet wall cross-section with location of Zone 5 filter and aggregate base course for paving.

interfaces: (1) embankment dam core to Zone 5 filter; and (2) Zone 5 filter to aggregate base course.

5.3.2.1 Filter check for Interface 1

Because the seepage during flood surcharge flows from the existing embankment dam core into the Zone 5 filter at Interface 1, the existing embankment dam core material functions as the base soil and the Zone 5 material functions as the filter for this filter check.

Step 1: Plot the gradation curves of the base soil materials and determine if the base soils have dispersive clay content. The gradation curves for the existing embankment dam core material are plotted on Figure 5-11. The gradation for the five samples is fairly uniform, with the gradation curves falling within a 10-point band for percent passing along the entire gradation curve. The existing embankment dam is located in a region that is not known for dispersive clays.

Step 2: Determine if the base soil has particles larger than the No. 4 sieve and if the base soil is gap-graded or potentially

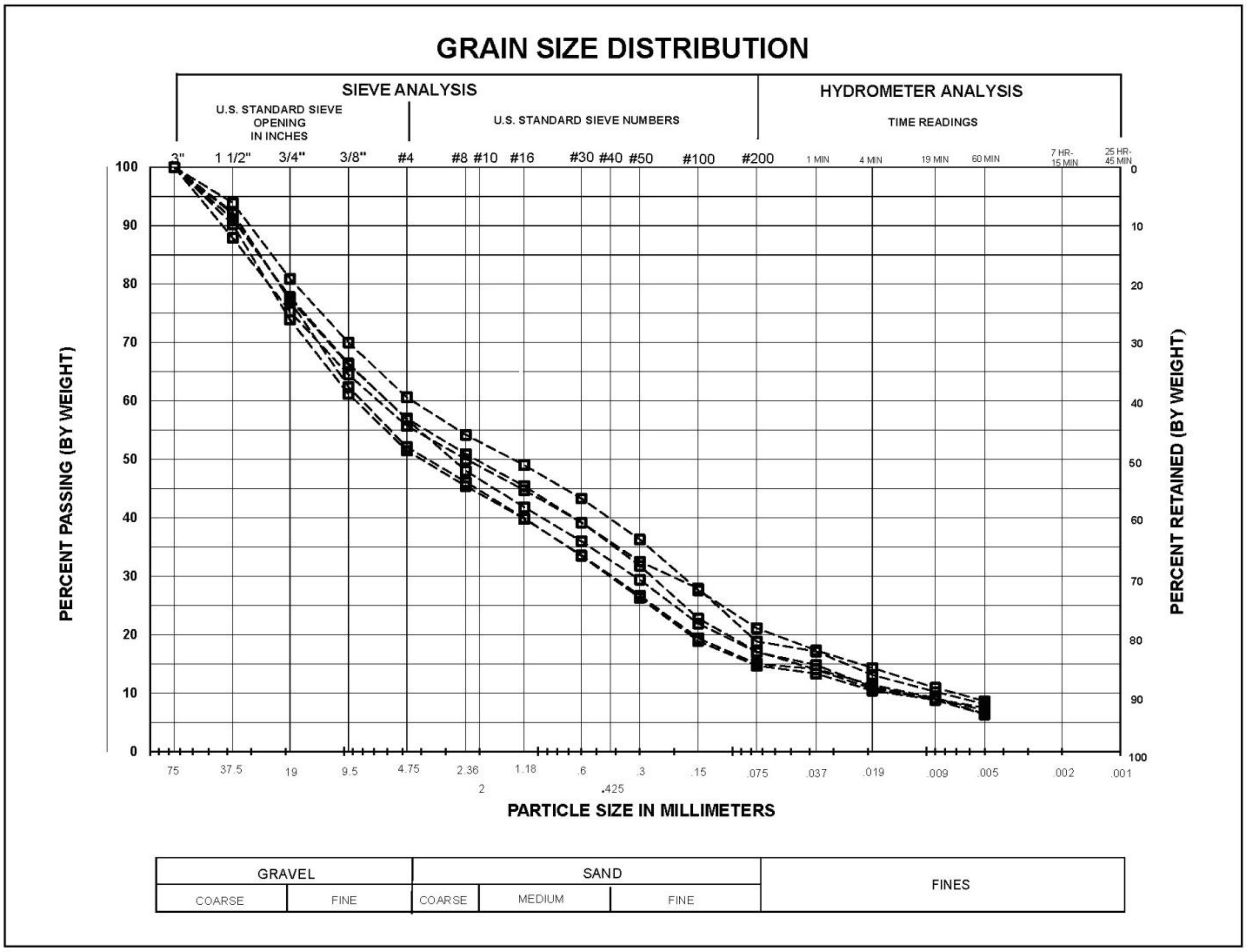

Figure 5-11. Existing embankment dam core gradations before re-grading.

subject to internal instability. The existing embankment dam core gradation curves include gravel contents in excess of 40% and fines contents of 15 to 20%. The soil is also broadly graded, with C_u = 398 to 811 (much greater than the limit of $C_u < 6$) and C_z = 0.64 to 1.57 (within the broadly graded range of 1 to 3). The gradation curves should be computationally re-graded.

Step 3: Prepare adjusted re-graded gradation curves for base soils. Each of the five gradation curves were re-graded using the procedure described in Chapter 5. The re-graded gradation curves are shown on Figure 5-12.

Step 4: Determine the base category of the soil based on the percent passing the No. 200 sieve in accordance with Table 5-1.

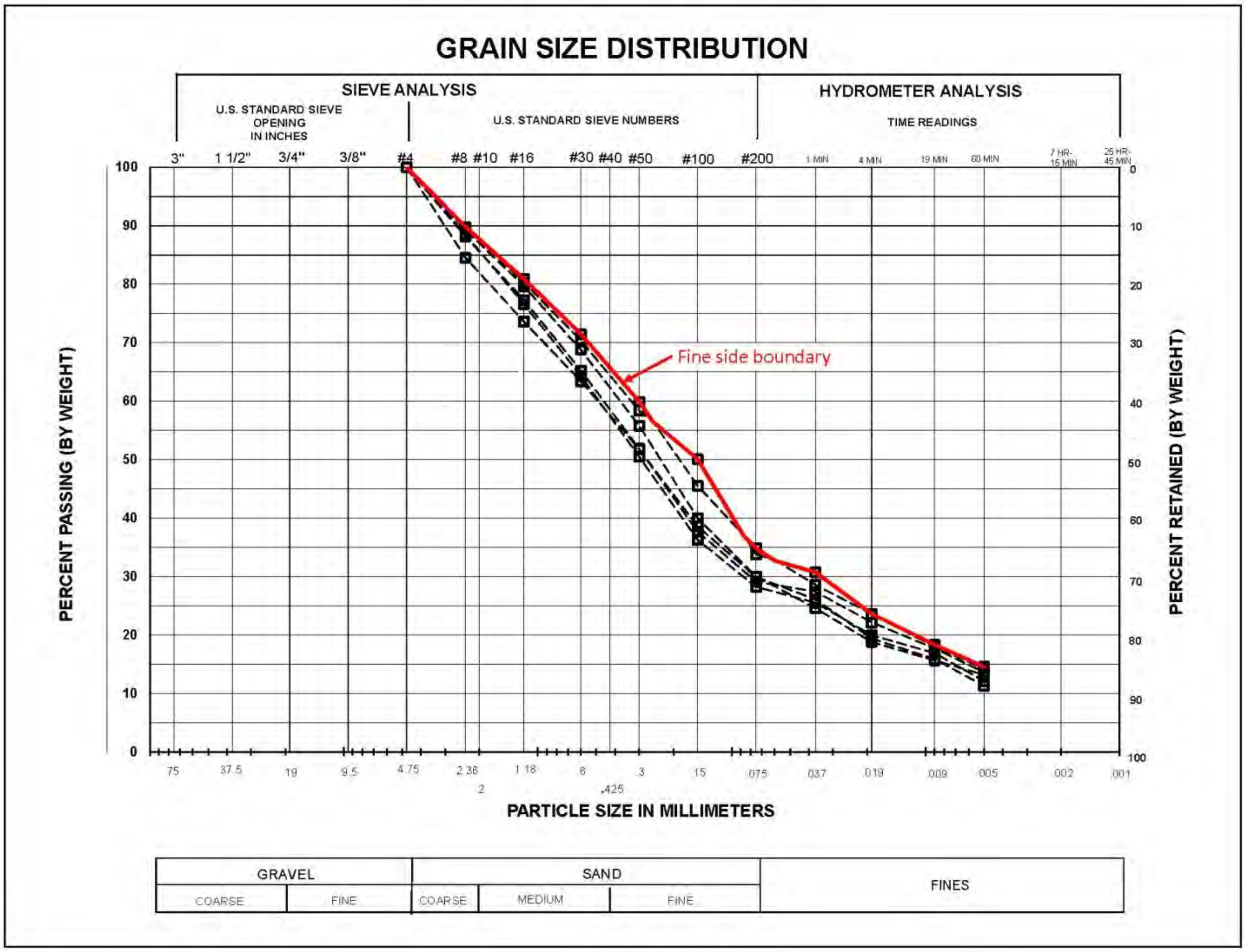

Figure 5-12. Existing embankment dam core gradations after re-grading.

The percent passing the No. 200 sieve for the re-graded curves fall in the range of 28 to 35%, resulting in a base soil category of 3 for all five gradation curves. Based on the guidance provided in Attachment A for base soil selection of earthfill materials with base soils that fall within one category for an existing dam (Figure A-14), the fine side boundary of the base soil gradation curves, as shown on Figure 5-12, should be used for filter design.

Step 5: Determine the maximum allowable $D_{15}F$ size to satisfy particle retention requirements in accordance with Table 5-2. For base soil category 3, with a fines content of 35% and $D_{85}B$ = 1.71 mm

from the fine side boundary of the existing embankment dam core gradation curves, the maximum $D_{15}F$ is calculated as:

$$(D_{15}F)_{max} = [(40-35)/(40-15)][(4)(1.71\text{ mm})-0.7\text{ mm}] + 0.7\text{ mm} = 1.98\text{ mm}$$

This value is plotted as filter control point A on Figure 5-13.

Step 6: Determine the minimum allowable $D_{15}F$ to satisfy permeability requirements. With $D_{15}B$ = 0.005 mm from the fine side boundary of the existing embankment dam core gradation curves, the equation for the minimum allowable $D_{15}F$ gives:

$$(D_{15}F)min = (5)(0.005mm) = 0.025\ mm$$

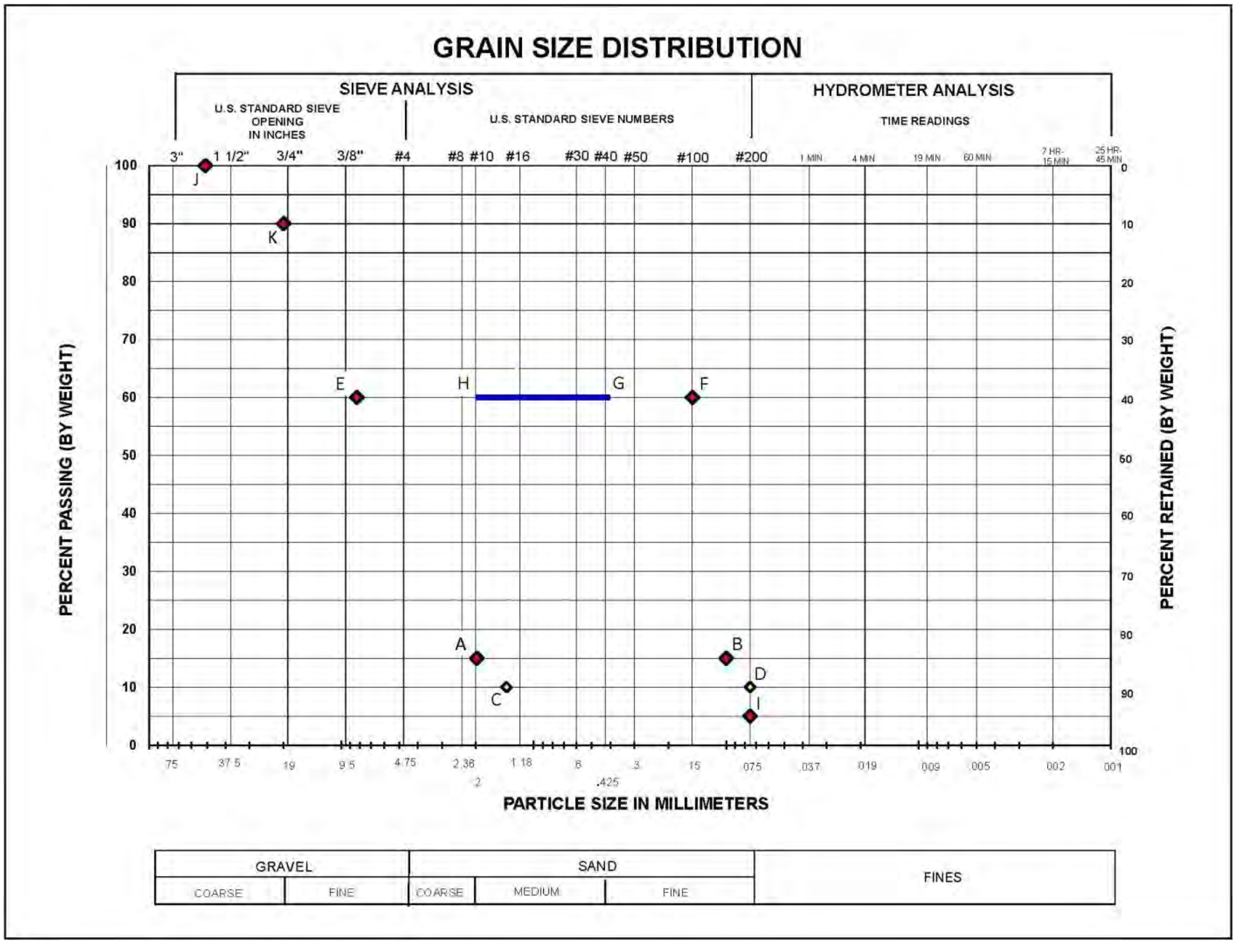

Figure 5-13. Filter control points for Interface 1.

This values is less than the minimum value of 0.1 mm specified in the procedure, so the minimum $D_{15}F$ = 0.1 mm. This value is plotted as point B on Figure 5-13.

Step 7: Determine the limits of $D_{60}F$ to limit the width of the filter band and possible gap-gradedness. In accordance with the guidelines provided in Chapter 5:

1. Maximum D_{10} anchor point (point C):

   $C = A \times 0.7 = (1.98 \text{ mm})(0.7) = 1.39 \text{ mm}$

2. Minimum D_{10} anchor point (point D):

   $D = B \times 0.7 = (0.1 \text{ mm})(0.7) = 0.07 \text{ mm}$, which is less than the minimum value of 0.75 mm

   Because the calculated value of D is less than the minimum value of 0.75 mm provided in the guidelines, D = 0.075 mm

3. Maximum D_{60} anchor point (point E):

   $E = C \times 6 = (1.39 \text{ mm})(6) = 8.34 \text{ mm}$

4. Minimum D_{60} anchor point (point F):

   $F = D \times 2 = (0.075 \text{ mm})(2) = 0.15 \text{ mm}$

5. The size of the sliding bar (points G & H):

   $G \geqq= 0.15 \text{ mm}$

   $H = G \times 5$

These values are plotted as points C through G on Figure 5-13.

Step 8: Determine the minimum D_5F and maximum $D_{100}F$ to limit the amount of fines and oversized material in accordance with Table 5-3. For all base soil categories, (D_5F) min = 0.075 mm and $(D_{100}F)$ max = 51 mm. These points are plotted as points I and J, respectively, on Figure 5-13.

Step 9: Determine the maximum $D_{90}F$ to limit segregation potential from Table 5-4. For all base soil categories, with a minimum $D_{10}F$ = 0.075 mm, the maximum $D_{90}F$ = 20 mm. This point is plotted as point K on Figure 5-13.

Step 10: Determine the gradation band within the control limits. As a trial, the gradation band for C33 "concrete sand" is plotted on Figure 5-14 along with the filter control points from Figure 5-13 to determine if it falls within the control points.

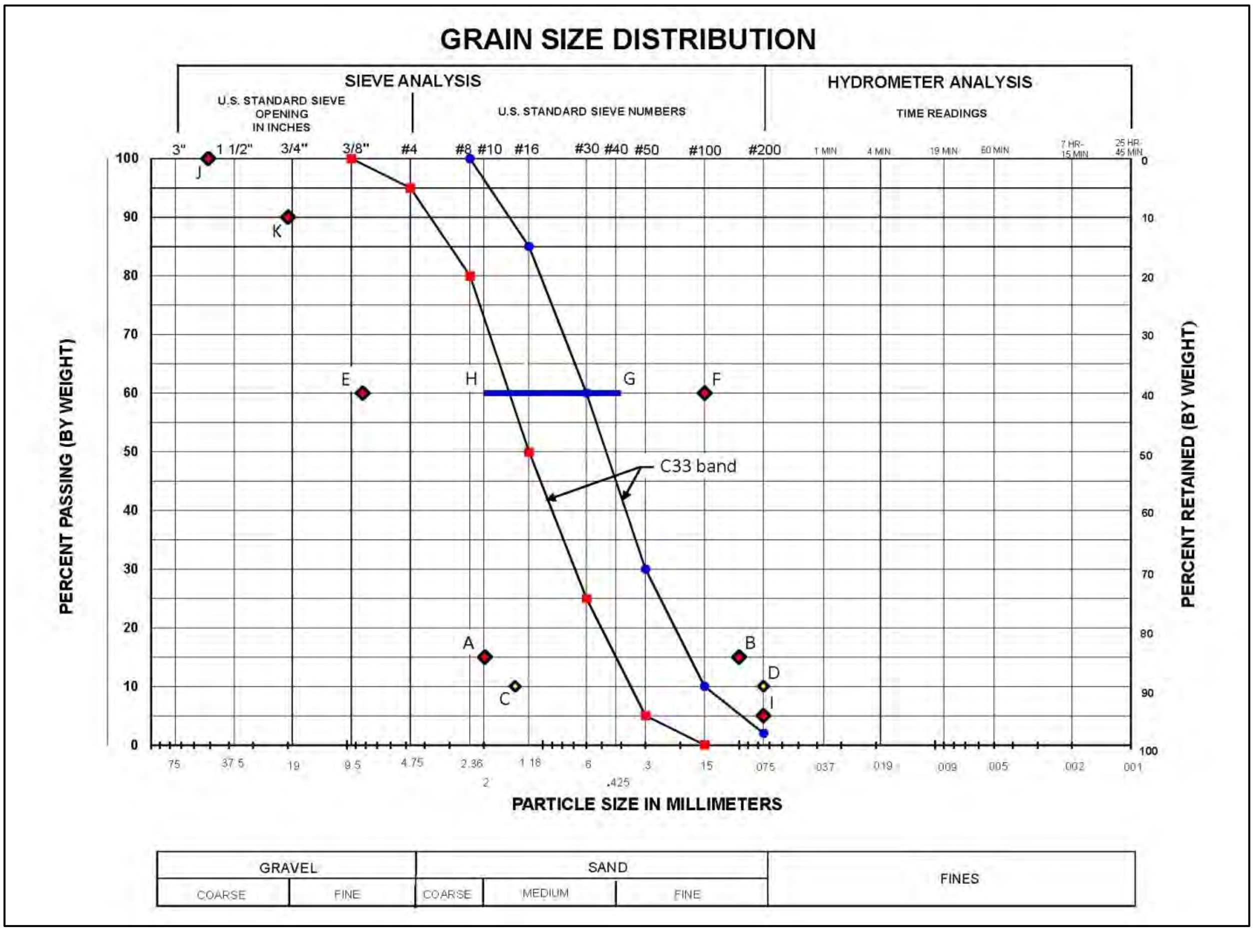

Figure 5-14. Gradation for C33 "concrete sand" plotted with the filter control points for Interface 1.

The band width defined by points G and H was slid between points E and F such that it coincides with the gradation band for C33 "concrete sand." Because the gradation band for C33 "concrete sand" falls within all of the filter control points for Interface 1, C33 "concrete sand" can be used as the filter material for this interface.

Alternate method for limiting gap-graded gradation during filter design: An alternate method for controlling the width of the mid-portion of the gradation band, based on the guidance provided in Attachment D, is shown on Figure 5-15. This method uses filter design control points A, B, I, J, and K from Figure 5-13, with a sliding vertical band defined by points L and M, that cannot cross the line between points A and K and requires the filter gradation to be no greater than 35 points. The gradation band for C33 "concrete sand" is also plotted in Figure 5-15 to check its compatibility with the filter design criteria. Because the gradation band for C33 "concrete sand" falls within all of the filter control points for Interface 1, C33 "concrete sand" can be used as the filter material for this interface.

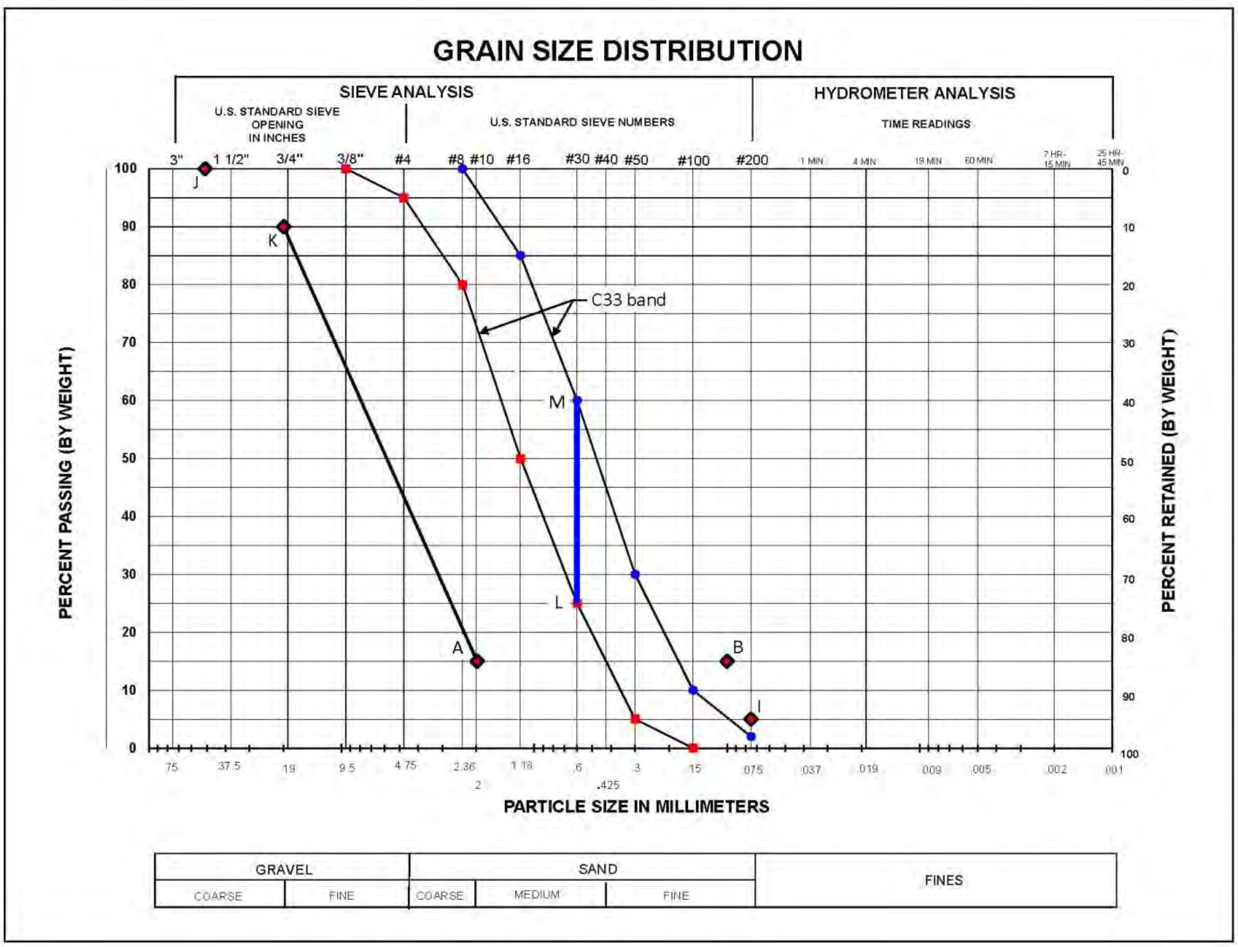

Figure 5-15. Gradation for C33 "concrete sand" plotted with the filter control points for Interface 1 from Alternate Method.

5.3.2.2 Filter Check for Interface 2

Because the seepage during flood surcharge would flow from the Zone 5 filter material into the aggregate base course for the asphalt paving at Interface 2, the C33 “concrete sand” filter material functions as the base soil and the aggregate base course functions as the filter for this filter check. The aggregate base course is an ASTM D448 No. 467 aggregate. The gradation is illustrated on Figure 5-16.

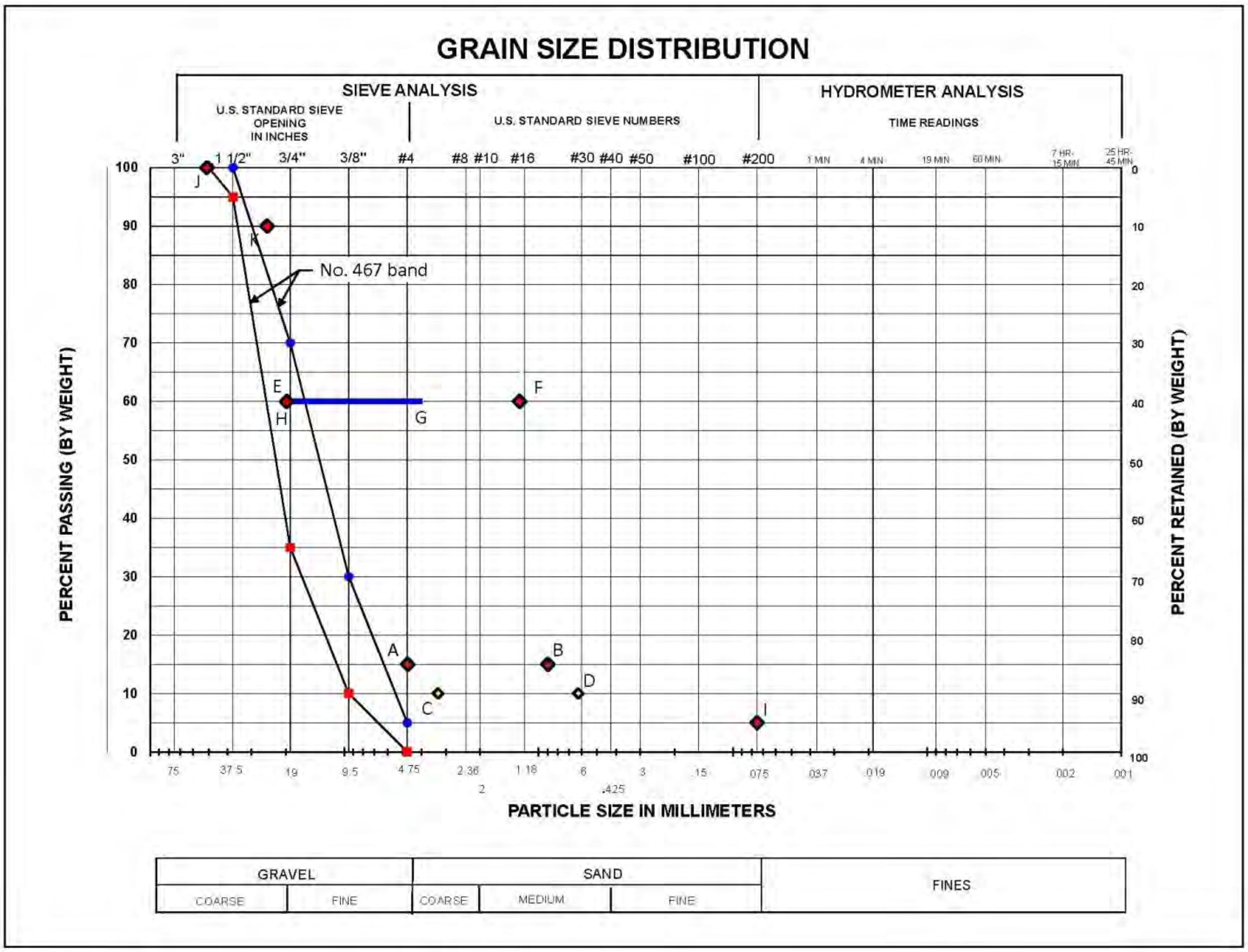

Figure 5-16. Gradation for ASTM D448 No. 467 plotted with the filter control points for Interface 2.

Steps 1-3: The gradation range for the C33 “concrete sand”, shown on Figures Figure 5-14 and Figure 5-15, is fairly uniform, and has less than 5% passing the No. 4 sieve. This material is not gap graded (C_u = 4 to 4.2 and C_z = 0.9 to 1.0). Therefore, the C33 gradations do not need to be re-graded.

Step 4: The percent passing the No. 200 sieve for C33 "concrete sand" is less than 2%, resulting in a base soil category of 4. Based on the guidance provided in Attachment A for base soil selection of earthfill materials with base soils that fall within one category, the fine side boundary should be used for filter design.

Step 5: For base soil category 4, with a $D_{85}B$ = 1.18 mm from the fine side boundary of the C33 gradation curves, the maximum $D_{15}F$ is calculated by:

$$(D_{15}F)_{max} = 4 \times D_{85}B = 4(1.18 \text{ mm}) = 4.72 \text{ mm}$$

This value is plotted as filter control point A on Figure 5-17.

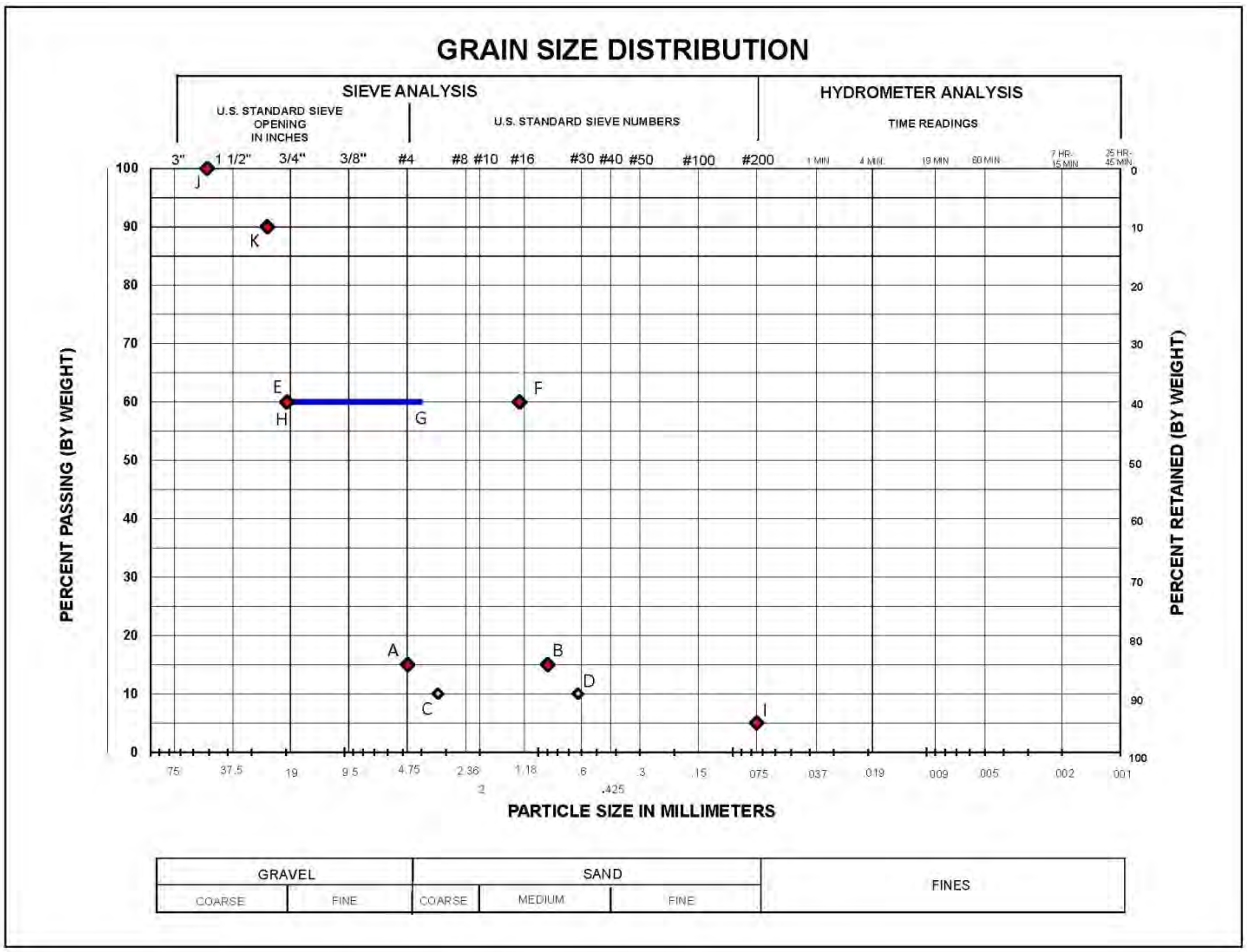

Figure 5-17. Filter Control Points for Interface 2.

Step 6: With $D_{15}B$ = 0.18 mm from the fine side boundary of the C33 gradation curves, the equation for the minimum allowable $D_{15}F$ gives:

$$(D_{15}F)_{min} = (5)(0.18\text{ mm}) = 0.9\text{ mm}$$

This value is plotted as point B on Figure 5-17.

Step 7: In accordance with the guidelines provided in Chapter 5:

1. Maximum D_{10} anchor point (point C):

   $C = A \times 0.7 = (4.72\text{ mm})(0.7) = 3.30\text{ mm}$

2. Minimum D_{10} anchor point (point D):

   $D = B \times 0.7 = (0.9\text{ mm})(0.7) = 0.63\text{ mm}$

3. Maximum D_{60} anchor point (point E):

   $E = C \times 6 = (3.30\text{ mm})(6) = 19.8\text{ mm}$

4. Minimum D_{60} anchor point (point F):

   $F = D \times 2 = (0.63\text{ mm})(2) = 1.26\text{ mm}$

5. The size of the sliding bar (points G & H):

   $G > 0.15\text{ mm}$

   $H = G \times 5$

These values are plotted as points C through G on Figure 5-17.

Step 8: For all base soil categories, (D_5F) min = 0.075 mm and ($D_{100}F$) max = 51 mm. These points are plotted as points I and J, respectively, on Figure 5-17.

Step 9: For all base soil categories, with a minimum $D_{10}F$ = 0.63 mm, the maximum $D_{90}F$ = 25 mm. This point is plotted as point K on Figure 5-17.

Step 10: ASTM D448 No. 467 is selected as a trial gradation. The gradation band for the No. 467 material is plotted on Figure 5-16 along with the filter control points from Figure 5-17 to determine if it falls within the control points.

Because the gradation band for the No. 467 material falls outside of the coarse side filter gradation control points (particle retention requirements) for the C33 "concrete sand," the filter design for this interface will be adjusted to emphasize permeability requirements, as discussed in Attachment A. For this filter interface, the maximum $D_{15}F$ can be increased to $(D_{15}F)$ max = 9 × $D_{85}B$, which will allow for particle rearrangement[1]. This is allowable since both the base soil (C33 "concrete sand") and the filter (ASTM D448 No. 467) are processed materials and grain size variability is minimized.

The adjusted maximum $D_{15}F$ (filter control point A):

$$(D_{15}F)_{max} = 9 \times D_{85}B = 9\ (1.18\text{ mm}) = 10.62\text{ mm}$$

The adjusted maximum D_{10} anchor point (point C):

$$C = A \times 0.7 = (10.62\text{ mm})(0.7) = 7.43\text{ mm}$$

The adjusted maximum D_{60} anchor point (point E):

$$E = C \times 6 = (7.43\text{ mm})(6) = 44.6\text{ mm}$$

In addition, the location of point K can be adjusted to consider a revised minimum $D_{10}F$ based on the No. 467 material being used as the filter material for this interface, rather than basing it on the filter control point D. For all base soil categories, with a minimum $D_{10}F$ = 5.5 mm from the fine side boundary of the No. 467 gradation, the maximum $D_{90}F$ = 50 mm.

---

[1] Also known as partial erosion, this is the erosion boundary between "no erosion" and "continuous erosion".

The adjusted points A, C, E, and K are plotted, along with the other filter control points for Interface 2 and the gradation band for ASTM D448 No. 467 on Figure 5-18.

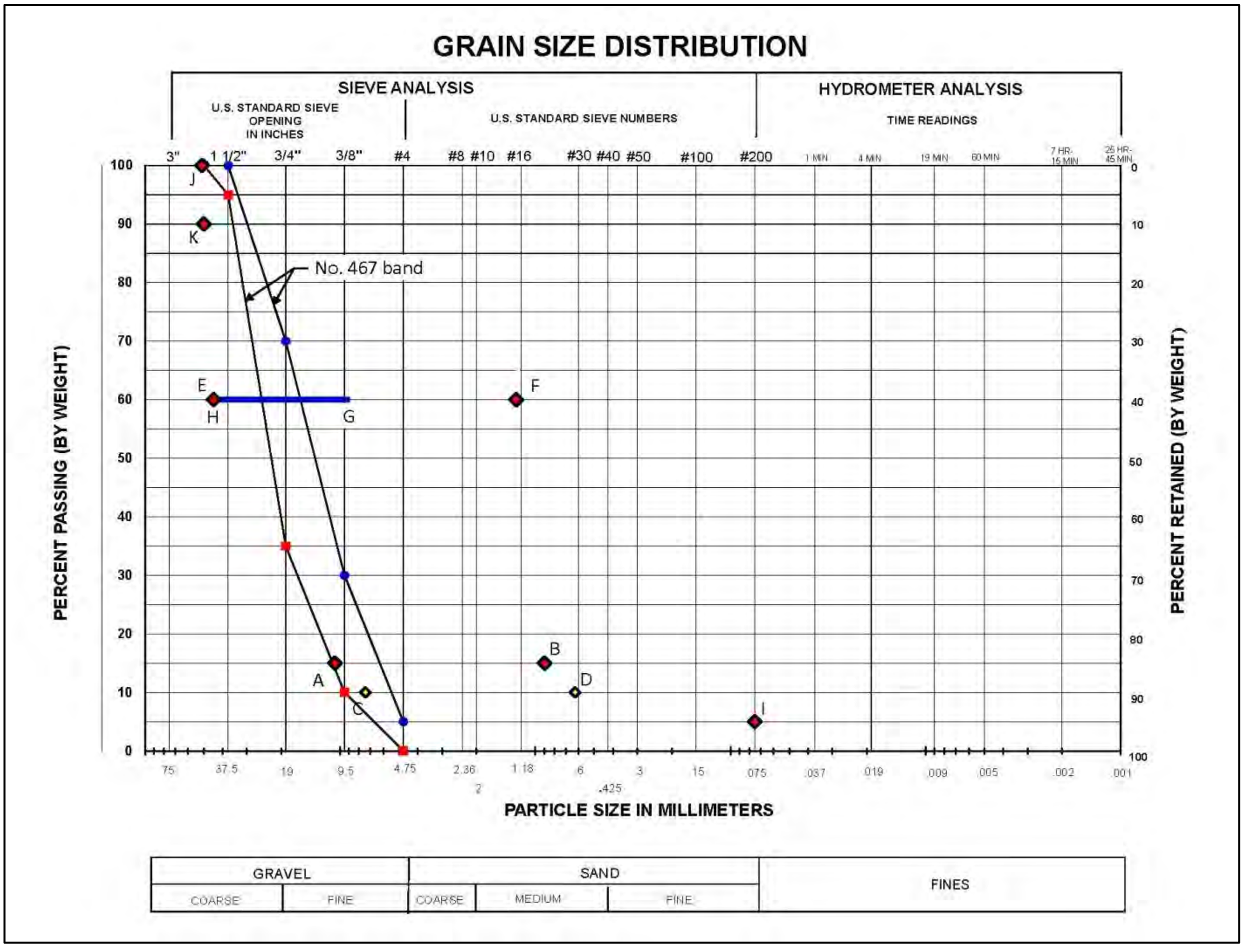

Figure 5-18. Gradation for ATSM D448 No. 467 material plotted with modified control points for Interface 2 (allow for particle rearrangement).

The band width defined by points G and H was slid between points E and F such that it coincides with the gradation band for No. 467. Because the gradation band for No. 467 falls within all of the filter control points for Interface 2, No. 467 is acceptable as the filter material for this interface.

5.3.2.3 Final gradations

The re-graded gradation curves for the existing embankment dam core material are plotted together with the gradation bands for the C33 "concrete sand" for the Zone 5 filter and the ASTM D448 No. 467 aggregate base course for paving on Figure 5-19.

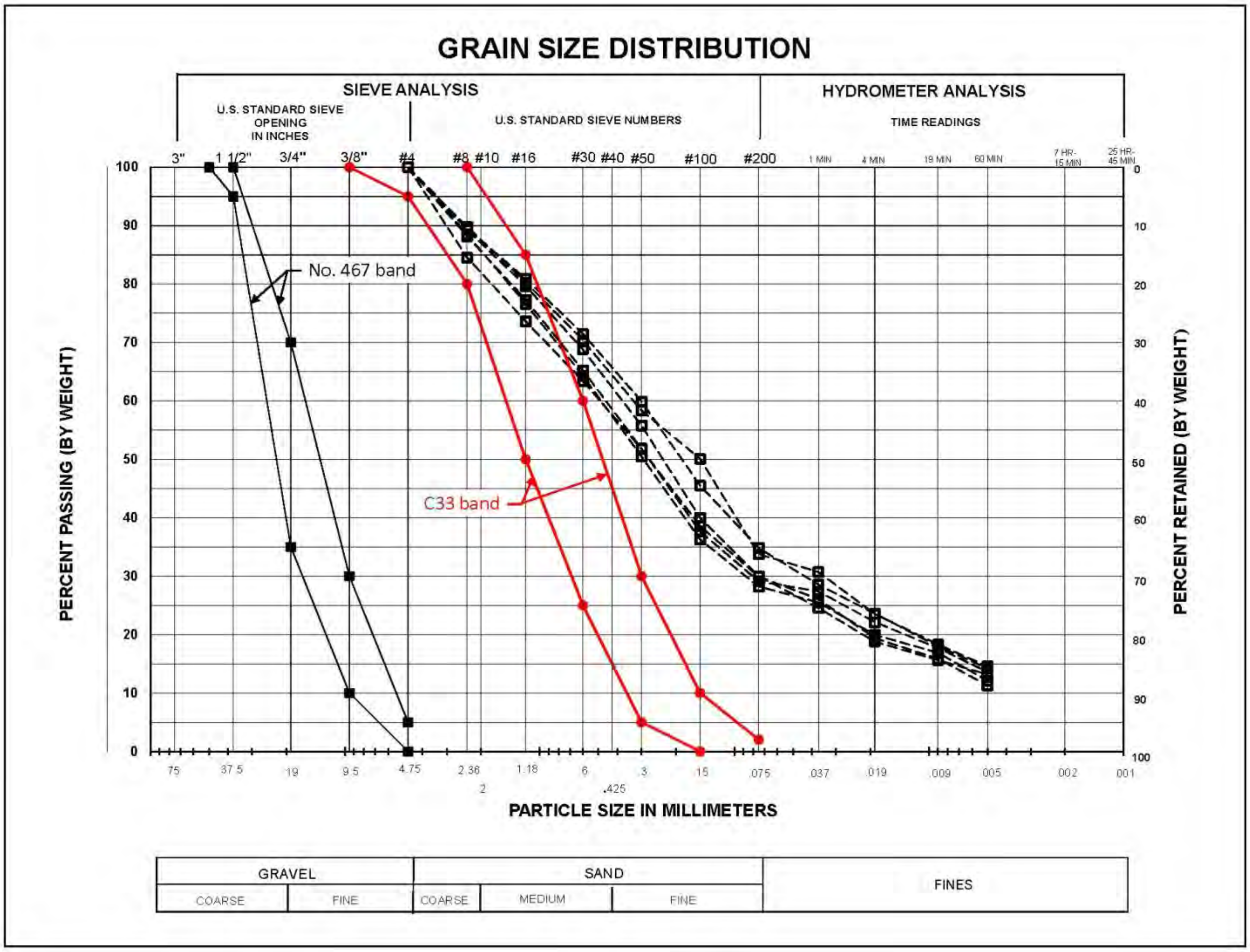

Figure 5-19. Gradations for re-graded existing embankment dam core material, C33 "concrete sand" (Zone 5 filter) and ASTM D448 No. 467 (aggregate base course).

6 Other Design Considerations

6.1 Introduction

This chapter will present additional design considerations primarily related to practical application of filter construction and the associated costs. In recent years, issues have arisen on the cost of filters used in dams, with the focus on smaller projects. As described in section 6.10.2, there is a balance between providing a quality product at a reasonable cost.

Specific topics covered in this chapter include critical gradient, minimum chimney width, minimum blanket thickness, areal extent (footprint) of filter usage, and cost issues related to material source location.

6.2 Critical gradient

In classical soil mechanics, a mathematical relationship defines the critical hydraulic gradient as the ratio between soil mass and the buoyant force of water acting against it. This simple relationship assumes that flow is upward and gravity must be overcome in order to move individual soil grains. This relationship assumes there are no intergranular forces present and the soil particles behave as spheres. The mathematical relationship is (see Symbols section):

$$I_c = \frac{\gamma_b}{\gamma_w} = \frac{Gs - 1}{1 + e} \text{ (upward flow)}$$

For a horizontal exit, the critical gradient can be determined by:

$$I_n = \tan \Phi \bullet I_c \text{ (horizontal flow)}$$

This is an accurate assumption for clean sand but not many other soil types. Unfortunately, this simple relationship has led to a misconception that critical gradient for any soil is unity. Testing results, discussed later, have indicated critical gradients can be an order of magnitude less than one.

Hydraulic gradient is the parameter most often used to judge whether sufficient seepage forces exist to detach and transfer soil grains. While it is not the most accurate predictor for soil erosion and is not always easily determined, it is the best predictor at this time. The actual ability of water to erode soil is a complex relationship between tractive forces present in the water flow and intergranular bonds between soil particles (soil strength and cohesion). That topic is beyond the scope of this manual; therefore, the gradient concept will be used for evaluation of erosion potential. It should be noted that soil type has as much impact on particle erosion as gradient. That is, highly plastic clay can sustain much higher gradients than a fine sand prior to particle movement. There are no known studies that address critical gradient as a function of material type.

Gradient must exist in order for flow through soil to occur. That gradient will vary from location to location within the soil mass since the seepage path length will vary from location to location. While gradient may be sufficient to cause the movement of soil particles within the soil mass, it is of little consequence since the particles typically will rearrange themselves into a stable configuration. However, concern does exist at material boundaries, such as those seen in embankment dams or at seepage exit boundaries (ground surface). The material boundary condition is described in Chapter 1, and a case is made for the need for filter protection. This section will describe the concept of critical gradient at an exit boundary and the issues associated with estimation of this parameter.

As described above, the calculation is based solely on overcoming gravity in the vertical direction. In cases where flow discharges from a vertical face or even downward, as into a pipe, the critical gradient would be much less. Recognizing this, factors of safety as large as four have been applied in design criteria. Still, research into the concept of critical gradient and appropriate values for use in analysis and design has been difficult to find.

Probably the most significant contribution to the subject has been in the work of Schmertmann (2001). That research focused on clean sands, but nevertheless, the work and evaluation method is about the best that is currently available. One of the most important conclusions of the research was that for worst case conditions (fine sands with a roof), the minimum hydraulic gradient that lead to material movement was only 0.08.

Schmertmann (2001) compared water velocities required to initiate erosion for two soils that were tested for both scour (in open channel flow) and piping. The results showed the velocity required for scour was 40 to 90 times that required for piping. He explained it by 3-D seepage modeling at the "pipe head" and found that the soil at that point could be in a "quick" condition, resulting in localized sloughing and transport under essentially zero effective stress. The tests were fairly conclusive, indicating that it takes much smaller velocities to move soil in a piping situation than in the bottom of an open flow channel. Nevertheless, a certain velocity and flow are needed to carry particles away from the pipe head once detached.

This work is also reinforced by a case history in which the critical gradient was back calculated from an active piping condition. Post-incident analysis of the piping condition seen at A. V. Watkins Dam (Reclamation, April 2007) showed that material was eroded in gradients of approximately 0.08.

It should be noted that research conducted by both Schmertmann (2001) and Geo Delft (J. B. A. Weijers and J. B. Sellmeigher 1993) through small- and large-scale piping tests indicated that low gradients could initiate piping (lower than the gradient calculated by Terzaghi's classical equation shown above). Both Schmertmann and Geo Delft noted that particle uniformity and particle size play a major role in critical gradients and that less gradient is needed for piping to occur with more uniformly graded and smaller particle sizes. This coincides with the occurrence of piping at A. V. Watkins Dam and Herbert Hoover Dike in which the material being piped was a very fine uniform sand. It appears that the more well graded the material is, some self-filtering action occurs and more gradient is need for piping. It is speculated that the larger particles are filtering the smaller particles (bridging), and higher gradients are needed to move the larger particles. In fact, for Schmertmann's laboratory piping models, he could not get materials with a $C_u > 6$ to pipe even when gradients were greater than 1, as shown in Figure 6-1. Note that this is based on his small-scale laboratory piping tests and that several corrections would be needed to correlate to field conditions.

Based on recent research, case histories, and known shortcomings of the classic view of critical gradient, values as low as 0.08 should be used for non-plastic silts, fine sands, silty sands, and sandy silts.

6.3 Minimum thickness of filter and drain zones

While filters used in embankment dams have a theoretical minimum, this dimension is not used in design because construction considerations will

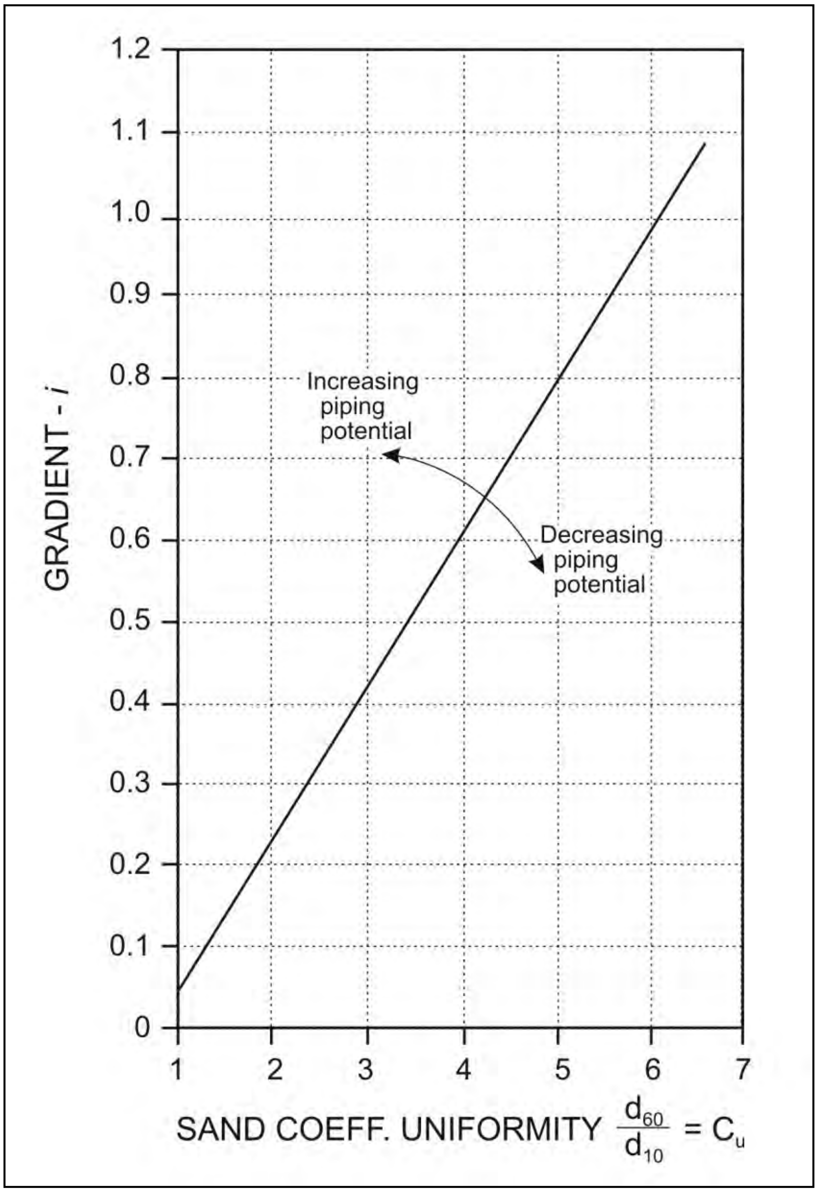

Figure 6-1. Horizontal piping gradient versus coefficient of uniformity.

control minimum thickness. Filters can be difficult to construct, and thin or nonexistent coverage will leave “windows” in the protection, rendering the filter useless. An example of this problem is presented in Attachment E, Case Histories – Narrow Toe Drain. For this reason, construction considerations are typically the deciding factors in specifying filter thickness. The special case of seismic offset may supersede filter width based on construction considerations. In seismically active areas, it may be possible that the dam will experience differential offsets of several feet. In cases where an embankment crosses an active fault, offset can be even more severe. The estimation of the magnitude of either type of offset is beyond the scope of this manual, but a conservative factor of safety for filter width should be used. Filter widths more than two times the maximum expected offset have been used in the past.

Once the minimum thickness based on construction considerations has been met, the thickness or width can be determined if the quantity of flow resulting from seepage or cracking is known. For major designs, this flow quantity can be computed by methods presented in Reclamation Design Standard, Seepage Analysis and Control (Bureau of Reclamation [Reclamation], 1987b) or methods presented by Cedergren (1989). For preliminary estimates of flow quantity and required thickness, the method developed by Justin (1945) produces satisfactory results. The width should be conservative so that a factor of safety is provided against unknown geotechnical conditions, inaccuracies in design parameters, deficient construction practice, etc.

6.4 Chimneys

Chimneys are inclined or vertical protective zones typically situated near the center of the embankment. The chimney connects to the blanket, described below, and as a minimum should extend above the top of active storage. Discharge capabilities of a chimney filter should be verified by suitable calculations and/or laboratory tests to ensure that they are capable of removing all water that reaches them without excessive head buildup, clogging, or piping of the filter itself (Cedergren 1973).

Three factors influence the width of vertical or inclined filters:

1. Orientation of the filter – vertical or inclined
2. Loading condition – static or seismic
3. Hazard classification – high, significant, or low
4. Ability to sustain a crack – thinner chimneys have a higher likelihood of sustaining a through crack.

Filter width is defined as the horizontal measurement across the filter. The filter thickness is defined as the measurement normal to the slope, sometimes known as the tangential thickness. Both definitions are illustrated in Figure 6-2. For vertical filters, the thickness equals the width.

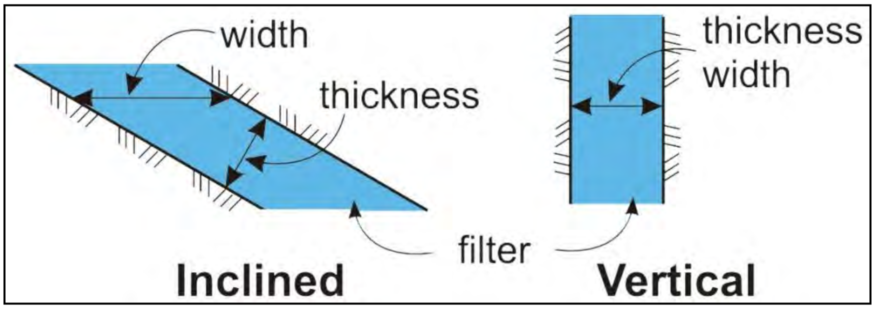

Figure 6-2. Definition of filter width and thickness.

When filters are placed against a slope, the width is always greater than the thickness. The difference between width and thickness increases as the slope becomes flatter, as shown in Figure 6-3. Narrow widths on flat slopes can lead to small thickness, which can be problematic due to the "Christmas tree" effect described later.

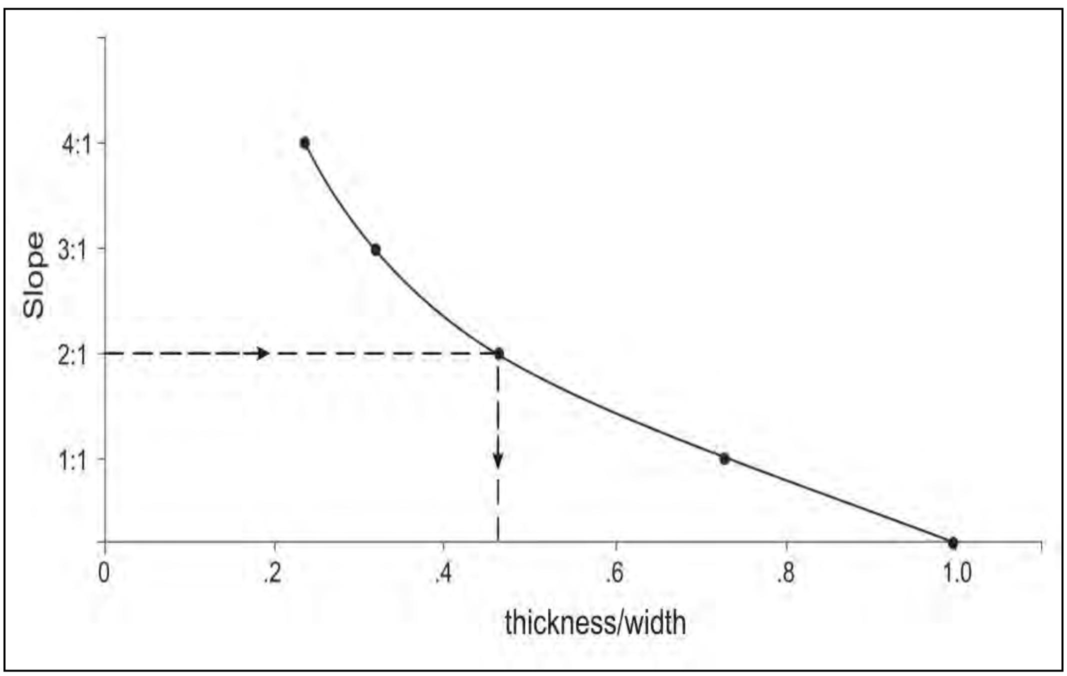

Figure 6-3. Effect of slope on filter width (e.g., a 10-ft-wide filter on a 2H:1V slope will have a 4.5-ft thickness).

When a filter is being designed to address seismic issues, the size of the filter is controlled by the maximum deformation expected from the seismic event. Deformations come from foundation fault displacement, foundation or embankment liquefaction in existing dams, and nonliquefaction settlement of the embankment or foundation. The filter size should be at least twice as large as the expected deformation (horizontal or vertical) in order to provide an adequate factor of safety.

When seismic protection is not required, filter width is typically controlled by construction methods. Since a variety of equipment is used for hauling and placement, and the size of that equipment is related to the size of the job, a variety of filter widths are found to be acceptable. Proven methods indicate that inclined chimneys can be reliably constructed at 6-ft and wider widths (Milligan 2003), and vertical filters can be reliably constructed at 4-ft and wider widths. Surveying and quality control/quality assurance are critical to ensure filter continuity.

Narrow zones require special placement procedures and intense inspection during construction. The crack resisting/self-healing capabilities of narrow zones are also less than for wide zones, and they should not be used if adequate materials are economically available. Often, reduced placement costs of wider zones will offset increased material quantity when narrow zones are contemplated. Cost considerations should only be the deciding factor when narrow zones meet the design requirements (hydraulic capacity, crack stopping, filtering, accommodation of postulated seismic movement, and self healing) adequately. Narrower filters can also become too thin when placed on flat slopes. Table 6-1 summarizes the filter thickness for a range of slopes and highlights filter width/slope combinations that result in a thickness of less than 2 ft (Pabst 2007c).

Table 6-1. Conditions in which filter thickness is less than 2 ft.

	Width – ft				
Slope	16	9	6	5	3
1:1	11.7	6.6	4.4	3.6	2.2
2:1	7.5	4.2	2.8	2.3	1.4
3:1	5.1	2.9	1.9	1.6	1.0
4:1	3.8	2.2	1.4	1.2	0.7

When narrow inclined zones are used, the designer should realize that placement procedures do not result in straight interfaces between filter drains and surrounding zones, but many have more of a “Christmas tree” appearance, as shown in Figure 6-4. The specified minimum width should account for the “Christmas tree” configuration to assume adequate drainage capacity.

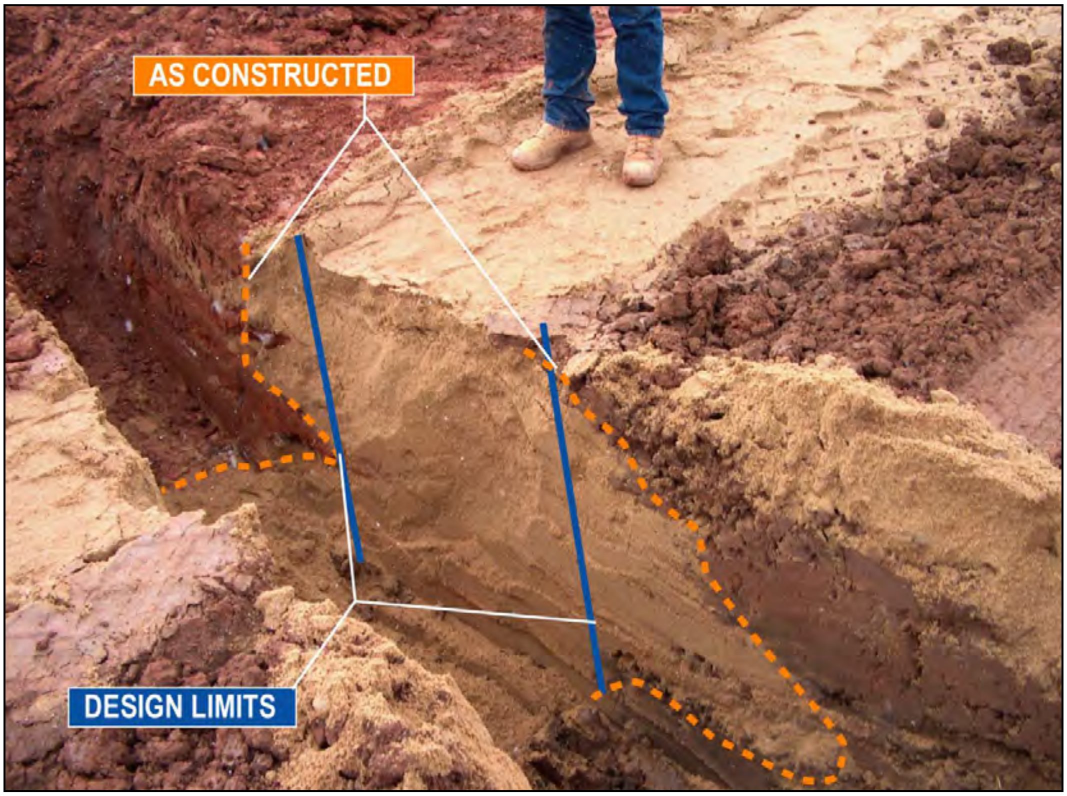

Figure 6-4. “Christmas tree” effect in a sloping chimney filter.
(Photo courtesy of URS Corp.)

Inclined chimneys also experience a reduction in width when lifts are not placed at a uniform elevation along the direction parallel to the axis of the dam. As the chimney is brought up, it is possible, and usually likely, that there will be low spots, or sags, along the top of the chimney. When a low area exists, a common mistake is to continue the lower portion parallel to the axis of the dam when that portion should actually shift downstream, and failing to make this correction will result in the chimney “thinning” out in the area of the sag. For a 2-ft sag on a 3H:1V slope, this can result in a 6-ft error. This error can result in the filter pinching out or leaving a window in the filter.

6.5 Blankets

Assuming that capacity requirements have been met, the minimum practical thickness of a drainage blanket is 18 in., with a desired thickness of 36 in. (Reclamation 2007). On steeper terrain or slopes, this may require special equipment and placement techniques as well as more intense inspection. When considering these concerns, the more prudent choice is often a thicker blanket.

6.6 Lateral and vertical extent of filter and drainage zones

In most cases, the vertical extent of filter protection in a dam (chimney) should be to the crest of the dam. Some designers prefer to end the chimney at the elevation of the maximum normal pool elevation, also known as the top of active conservation (TAC). This practice is also appealing due to difficulties in constructing a chimney in the narrowest portion of the embankment. The argument against this practice is that the most likely location of cracks in a dam is at the crest, so chimneys should be taken to that elevation. In cases where freeboard exists above the maximum flood pool elevation, to provide protection against wave run-up during maximum flood events, the chimney can be terminated at the maximum flood pool elevation.

The lateral extent of filter protection on abutments (blanket) is dependent on canyon or valley geometry and geologic conditions. For broad or wide valleys (gentle abutment slopes), the blanket should be extended up to the elevation of maximum normal pool (TAC). For cases where abutment slopes are steep, such as in canyons, the condition of the rock will dictate the extent of protection. For good quality rock with little fracturing, no protection is needed. For highly fractured rock where seepage conditions are expected to be large, blanketing is required. Note that in this situation, blanketing should be used regardless of the amount of foundation grouting or surface treatment.

Where chimneys intersect steep abutments or structures (concrete gravity sections, spillway walls, etc.), the chimney can be flared in order to increase its surface area on the abutment or section as described in section 2.3.4.

6.7 Angular versus rounded particles

Depending on the source, material for filters and drains can range from rounded to angular grains. Rounded material will generally be found in areas where pit run sands and gravels are screened and washed to produce aggregates. This material is generally taken from alluvium found along current or ancient water courses. Angular-shaped particles typically come from crushing operations in which the source material is either rock that is removed by blasting or oversized cobbles and boulders. Crushed particles often contain excess fines that may cement the filter once placed. Crushed materials should always be washed to remove fines.

Generally, rounded particles have greater crushing resistance than angular particles; but, strength is not a critical characteristic for filters and drains. Similarly, rounded particles will have favorable flow characteristics compared to angular particles, but the difference is negligible. Therefore, particle shape should not be a restriction when specifying filter material.

6.8 Uniformly graded versus broadly graded materials

Grain size distribution of any given soil will affect that soil's permeability. Generally, a uniformly graded soil will have a greater permeability than a broadly graded soil when they have the same D_{10} size. This is because void space between sand particles in the uniformly graded sand is replaced by gravel particles in the broadly graded mixture as shown in Figure 6-5. The left side of the figure illustrates spheres of two sizes representing a uniformly graded soil (example: coarse sand). On the right side of the figure, three larger spheres overlay original figure and are shown in red. They represent the inclusion of gravel-size particles, making the soil broadly graded. The figure illustrates that the larger particles now replace previously available seepage space through voids, and that lost space has been highlighted in blue. Note that the figure has not been corrected for the larger particle's edge to edge contact with the surrounding particles. The elimination of void space in the broadly graded soil results in a lower permeability (Pabst 2007).

6.9 Capacity for coarse foundations

Designing filter and drainage elements for coarse foundations can be problematic due to the many unknowns that exist even after extensive site

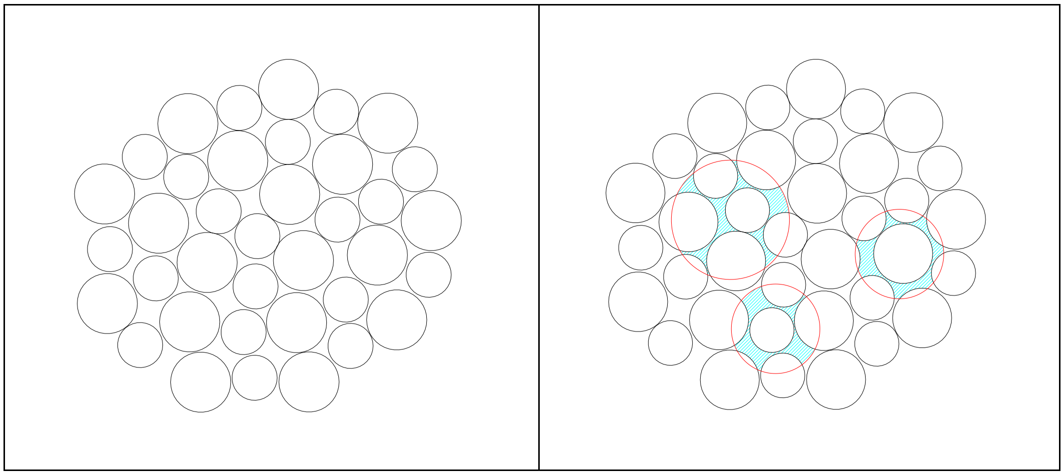

Figure 6-5. The illustration on the left shows idealized spheres of two sizes and resulting void space between the spheres. For the illustration on the right, three larger spheres (red) are overlain on the original illustration. This demonstrates how the larger spheres will replace once available void space, highlighted in blue.

characterization studies. Figure 6-6 shows a foundation of a dam built in 1920. While seepage performance and geologic exploration indicated a pervious foundation, it remained surprising the amount of open work observed after excavation. This problem is especially difficult for new dams since initial reservoir filling will be the first loading condition. Experience has shown that it is easy to underestimate seepage that flows through these types of foundations. Techniques for estimating these flows have changed over time mostly due to computational advancements. Whether the estimate is made by hand calculation or by computer, the material property assumptions will dictate whether or not an accurate prediction is made (Cedergren 1973).

Parametric studies should be performed assuming a range of permeability and anisotropy for the critical foundation materials (Pabst 2008). Since the best understanding of foundation conditions is not available until after excavation, the design should be based on the worst reasonable foundation conditions that can be expected.

6.10 Filter material sources

Borrow sources for filter material production can be from undeveloped sites or existing commercial sources. Undeveloped sources may or may not be within the project boundaries. On large projects in remote areas, filters may be manufactured by on-site crushing, screening, and processing to

Figure 6-6. Open work present in the right abutment foundation of Ochoco Dam. The abutment consists of landslide debris.

meet specified gradations. Commercial sources are almost always some distance from the work and can be as much as 100 miles away in some rural areas. Whether undeveloped or commercial sources are used is a question of economy. If no undeveloped site can be found closer to the work than a commercial site, the commercial site will be used. Even when commercial sites are the only opportunity for supply, several should be included in the solicitation so that the best price is achieved through bidder competition.

6.10.1 Identifying and investigating material availability

Local sources must be investigated and, for approved sources, appropriate information such as location, availability, ownership, drill logs, test pit logs, appropriate lab data, and geotechnical considerations provided in the specifications.

The first step in identifying undeveloped sources is to perform a literature review. Existing literature will be the quickest way to find possible borrow areas. These sources include "quad" sheets, soil reports, and regional geology reports. Quad sheets (U.S. Geological Survey [USGS] quadrangle maps) of the local area can be obtained and examined for existing or historic quarries, indicated by a mining symbol (a pair of crossed picks). Also available to the general public are Natural Resource Conservation Service (formerly Soil Conservation Service) soil reports (known as soil surveys). These reports, produced for almost every county in the United States, contain soil maps of the county as well as the engineering properties for those soils. While the soils are described in agronomic terms, the information is still valuable for engineers. Also available, but not uniformly produced, are Pleistocene (or Quaternary) geologic reports produced by the USGS, universities, and other interested parties. While these reports may not identify specific borrow area locations, they are useful for correlating stratigraphic geology assumptions and may indicate promising locations for closer examination.

After completing a literature review for the area, a terrain interpretation step should be undertaken. Terrain interpretation can be done two ways: by aerial photography and by site reconnaissance. Terrain interpretation of photographs is described in several text books (Hamblin 1982; Ritter 1978) where a description is given of changes of vegetation and land use that often indicate what soils are present.

During site reconnaissance, the typical way to identify soil profiles or other erosional features is by observing road cuts and naturally occurring cuts. Figure 6-7 illustrates a moraine that has been dissected by a creek and provides an early indication of the underlying stratigraphy.

In general, sand and gravel deposits are associated with the following geologic features:

- Alluvium along water courses
- Glacial outwash deposits
- Alluvial fans

Information included in the specifications must be adequate to allow bidders to develop reliable costs for preparing their bid. Borrow area information for approved borrow sources must be sufficient for the bidder to design the processing plant. The range of material gradation in the borrow area must be determined and this information clearly conveyed so the processing plant can be designed with sufficient flexibility to handle the

Figure 6-7. Exposed moraine cross section showing till overlying glacial outwash. Such exposures provide an opportunity to obtain geologic information without an expensive exploration program.

range of material sizes. Plants designed without this flexibility have been the cause of some large changed condition claims from contractors declaring that the information furnished was inaccurate or insufficient and misled them in their plant design.

Also critical for borrow area characterization are the groundwater conditions. Since excavation techniques will be different above and below the water table, a clear understanding of this level, and its fluctuations, is necessary. If dewatering is required for borrow area use, the cost will need to be factored into the project estimate. Consideration also needs to be given to seasonal fluctuations in water levels. Providing a single static level to bidders can result in a claim if the groundwater level rises later and floods out the contractor. Therefore, water level readings should be collected for the full range of expected water levels and presented in the specification.

Common exploration methods for borrow area studies include augering, trenching, and test pits. Which method is used is dependent on the maximum particle size of the material and material variability. Material that is smaller than 3 in. should be sent to the laboratory for gradation analysis.

Large material is typically estimated in the field visually. It is imperative that the full range of material sizes be presented in the specifications since history has shown claims are made on this critical characterization.

The preferred exploration method for sand and gravel borrow areas is trenching. Trenches are usually excavated using a utility tractor or track-hoe (excavator), although larger trenches may be excavated by a dozer. Initially, trench side slopes should always be vertical to give the best representation of the material. For safety reasons, personnel should not be allowed in vertical-sided trenches greater than 3 ft deep. Figure 6-8 illustrates a technique that can be used to excavate an exploration trench that can be entered for mapping and sample retrieval.

When sampling from trenches, all material should be collected, including oversized, so the percentage of oversized in the borrow can be estimated.

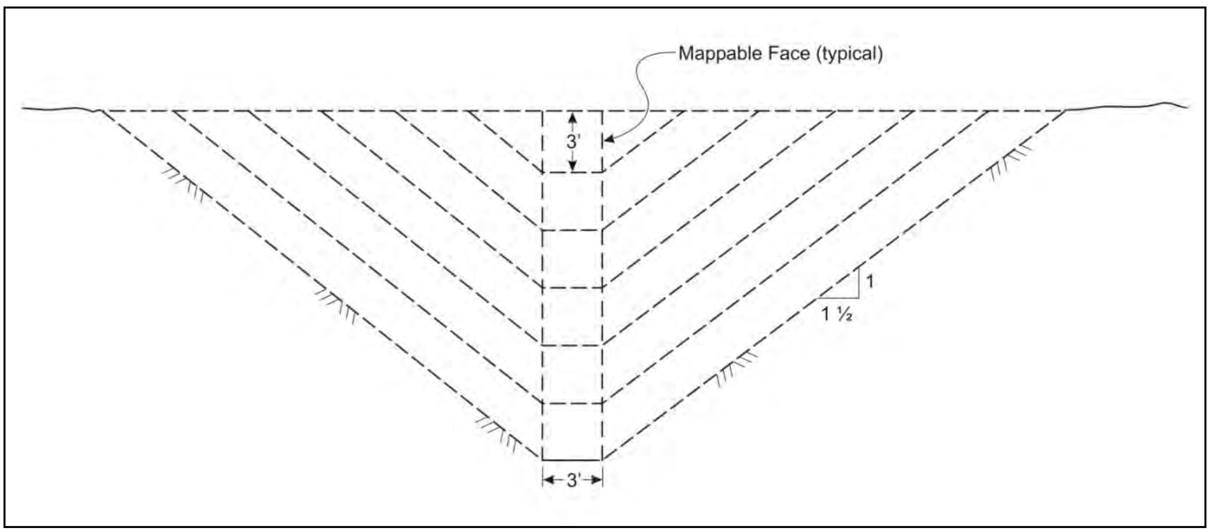

Figure 6-8. Exploration trench excavation sequence.

When boulders are present, their volume will have to be estimated visually. Figure 6-9 shows a trench excavation with the boulders set to one side of the trench, indicating the size and distribution of the boulders.
Figure 6-10 shows the material distribution in the trench wall, including interbedding. Note that this trench wall gives a much more detailed description of the materials than what would be obtained from drill hole data.

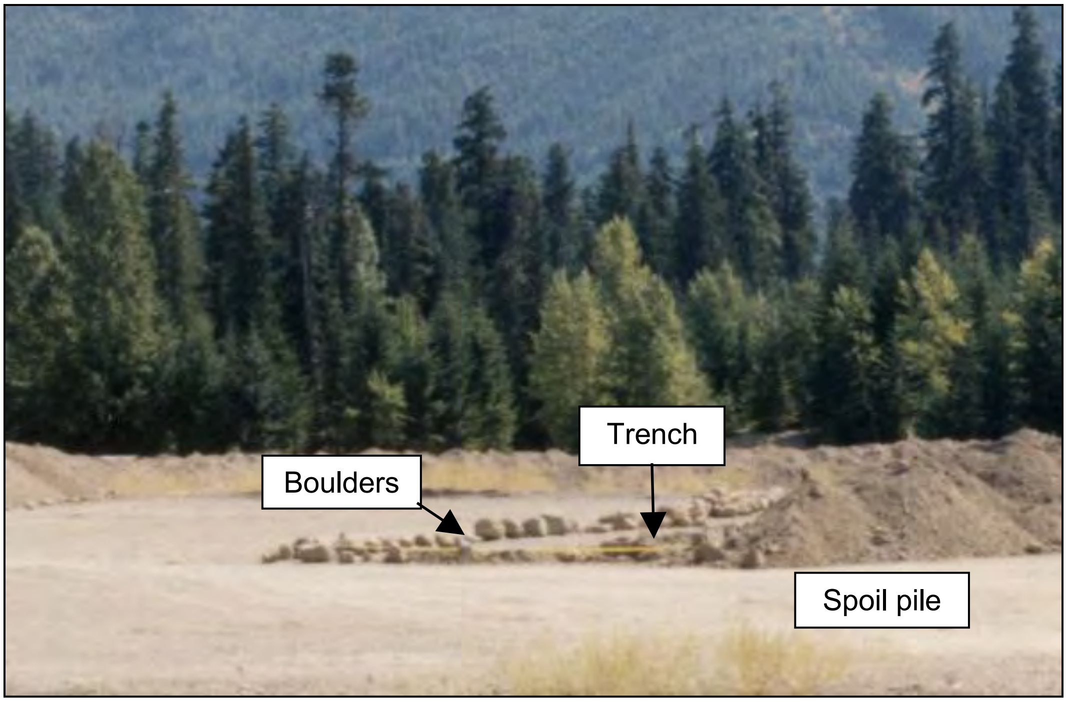

Figure 6-9. Exploratory trench excavated at a potential borrow area. During the excavation, the boulder-size material was set aside to better characterize the deposit.

The availability and suitability of material must be factored into the design. For example, if suitable material is limited in quantity or expensive to obtain, it may be more economical to use narrower zones requiring intensive inspection techniques. On the other hand, if ample material is near the job site and can be economically developed, equipment width dimensions of filter/drain zones with less intensive placement and inspection techniques may be more cost effective.

For undeveloped sources, ensure that there is a sufficient volume of material available for construction. Generally, it is prudent to have at least two to four times the volume of material available in borrow than is necessary to produce the final in-place volume of the filter/drain zones. For large jobs, a sieve-by-sieve analysis should be made in order to determine which grain size is critical for a specific pit. Attachment C demonstrates how such an analysis is made.

Consideration should also be given to the project schedule. Depending on land ownership and State regulation, a newly opened borrow pit may require one or more permits. The permitting process can be lengthy and may not be achievable within the project schedule. There will also be a cost

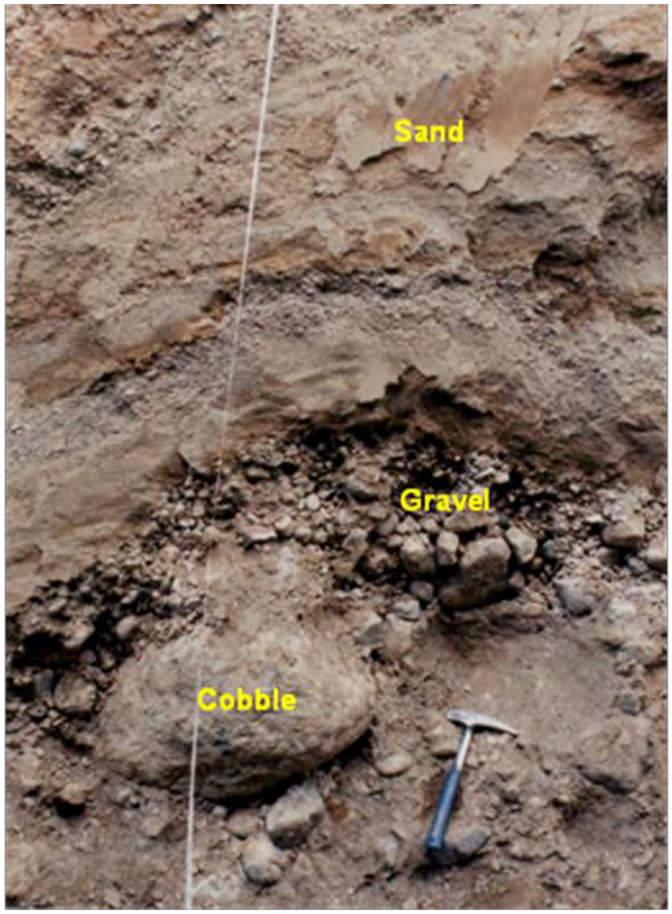

Figure 6-10. Exposed vertical trench face indicating the stratigraphy of a potential borrow area. This type of exposure provides a level of information not available by exploratory drilling.

associated with this work, which should be included in the analysis described above.

Typically, the higher the fines content of filter material, the greater the processing cost. This is due to the amount of processing needed to remove the fines. Typically, washing is required to remove the fines, and this operation is one of the most expensive procedures in the production of clean material. As described elsewhere in this manual, the amount of allowable fines depends on where and how the filter is going to be used.

Listed previously in Section 2.2, Reclamation (2007a) lists the following filter classes:

- **Drainage filters (class I)** – Filters whose purpose is to intercept and carry away the main seepage within a dam and its foundation. These filters may have to remove large amounts of seepage for dams on pervious foundations or dams of poor construction. The filters consist of uniformly graded materials, typically in two stages. The filter must meet the requirements for both particle movement and drainage. Toe drains typically fall into this class.

- **Protective filters (class II)** – Filters whose purpose is to protect base material from eroding into other embankment zones and to provide some drainage function in order to control pore pressure in the dam. These filters are typically uniformly graded and in several stages, but they can also be broadly graded in the interest of reducing the number of zones to make the transition to the base material. This class includes chimneys, blankets, and transition zones on the downstream side of a dam.
- **Choke(inverted) filters (class III)** – Filters whose purpose is to support overlying fill (the base material) from moving into pervious or open work foundations. These filters are typically broadly graded and have a requirement only to stop particle movement. There is no permeability requirement. Choke filter material is also used in emergency situations in an effort to plug whirlpools and sinkholes.
- **Seismic crack stoppers (class IV)** – Filters whose purpose is to protect against cracks that may occur in the embankment core, especially caused by seismic loading and/or large deformations. The dimensions of this class of filter are controlled by expected displacement (horizontal or vertical). While there is no permeability requirement for this type of filter, it should be relatively free of fines so the zone itself does not sustain a crack. A second stage (gravel) filter may be required if concern exists that the first stage finer zone might sustain or allow propagation of a crack. Second stage filters may also be required for transition to a coarser shell material. This class of filter is typically used for chimneys and transition zones.

A summary of these filter classes and their requirements is given in Table 6-2. The stage, gradation, and permeability issues are described in more detail in Chapter 2.

6.10.3 Lack of suitable clean materials

While sand and gravel soils are ideal for production of filters and drain materials, they are seldom found “clean” in situ. Usually, some amount of fine material (soil finer than the No. 200 sieve) will be present in the

Table 6-2. Filter classes and their uses and requirements.

Class	Filter Type	Uses	Multiple Stages Required?	Uniform Gradation Required?	Permeability/ Drainage Required?
I	Drainage	Toe drains, relief wells, drain fields	Yes	Yes	Yes
II	Protective	Downstream chimneys, blankets, transition zones	Sometimes	No	Yes
III	Choke	Foundation filters, sinkhole backfill	No	No	No
IV	Seismic crack stopper	Upstream and downstream chimneys	Sometimes	Yes	No (although the filter should be self healing and not sustain a crack)

deposits. Typically, the amount of fines present will define whether the pit is acceptable or not. Commonly available processing plants can economically process raw material with about 8% fines content (based on a sample with material greater than 3 in. in diameter removed). As described in Chapter 5, the fines content of the material that comes out of the plant (in stockpile) should not be greater than 5%. A number of washing operations are available, including spray bars, sand screws, sluice trays, etc., to remove these fines. These methods are successful when the fines are evenly distributed throughout the raw material. Borrow areas with layers of clay or silt may make the area unusable. The elevation and thickness of the layer or layers will influence whether or not a borrow area will be usable. A layer on the ground surface can be readily stripped and wasted prior to excavating the desired sand and gravel deposits. The limiting thickness of an overlying layer will be a function of the cost analysis described earlier. Layers throughout the pit are more difficult to analyze. Thin layers, less than 1 in. in thickness, may be acceptable if the blended fines content for the mass is less than 8%. That is, numerous 1-inch layers or a high percentage of 1-inch layers may make a borrow area unacceptable. Situations in which layers are several feet thick and at depth within the pit usually will render the pit unusable. Since pits are typically excavated from a vertical face, either from the top using a trackhoe (excavator) or from the bottom using a loader, the low quality layer will contaminate each load. In some instances, it may be possible to excavate a desirable layer and send it to the processing plant, excavate an undesirable layer to waste, and then return to excavating the underlying desirable material. This operation will have the added cost of either stockpiling the upper clean layer before feeding it to the processing plant or shutting down the plant while the undesirable layer is removed.

In addition to fines occurring in discrete layers, problems can also arise from fines adhering to larger particles such as gravel and cobbles. During borrow investigation, larger particles should be specifically examined for fines adhesion. As a general rule, fines that are easily wiped off of the larger particles by hand can be successfully washed in the processing plant. Fines that can only be removed with effort by rubbing cannot be cleaned by any type of washing operation. This condition is usually only found above the groundwater table, and similar material below the water table has a better likelihood of being washed. In this situation, the material above the water table may be unusable even if the fines content is less than 8%.[1]

Along with consideration of the amount of fines in a potential pit, the quality of the aggregate should also be determined. See chapter 4 for a description of the quality requirements for filter and drain materials and the test methods that can be used to meet these quality requirements.

6.10.4 Production plants for filter materials

Processing plants consist of three major operations: raking, screening, and washing. The raking operation removes all oversize material, typically material larger than 3 in. Raking can be done in the borrow area by running a rake through the excavation surface, which picks out the oversize material; at a loader with a skeleton bucket, which retains the oversize material; or at the initial feed into the plant through a feed box, which has a grate set to the desired size limit.

Screening operations consist of mechanical screening using a number of screen sizes dependent on the gradation of the borrow area and required materials. Screening is typically done in the dry, although spray bars may

[1] Laboratory gradation testing should always be done utilizing “wet” sieving while recognizing that the addition of sodium metahexaphosphate (wetting agent) will remove adhered fines—a procedure that cannot be duplicated at the processing plant.

be used in the interest of reducing dust. Similar to the raking operation, the larger sizes are separated out first in order to reduce wear on the finer screens.

The final operation is sand washing. Since raking and screening operations separate out the oversize and gravel sizes, only sand, silt, and clay remain at the far end of the plant. Separating the sand and fines (silt and clay) require wet washing. While a number of methods are commercially available, some proprietary, they consist of the same general concept, introducing the sand/silt/clay mixture into standing water and agitating. This "washing machine" effect permits the larger particles (sand) to go to the bottom of the mixer while the smaller particles (silt and clay) float to the surface or remain in suspension where they are drained off. The sand is then directed to a conveyor where it is stockpiled, whereas the silty clay slurry is delivered to settling ponds.

In areas where pit run material is not available but high quality rock is, the rock can be excavated by blasting and crushing to sand and gravel sizes. It should not be assumed that the crushed material is free of fines, and material obtained by this method should be washed as described above.

Since the plant separation process results in multiple stockpiles of gravel and multiple stockpiles of sand, these materials are blended back together to make the desired end product. The process, also known as reassembly, is typically a separate operation from the screening plant.

When insufficient or no sand is present in a borrow area, a crushing operation can be added to the processing plant (see Attachment C). In some parts of the country, bedrock is drilled, blasted, and crushed to sand and gravel sizes, although this is more costly than using naturally occurring sand and gravel deposits.

As described in previous sections, lead time to develop a borrow area and process the material can be long. To help offset some of this time, a "materials" solicitation can be produced prior to the solicitation for the major work. A "materials" solicitation can be produced relatively quickly, and a contractor can produce and stockpile material during preparation of the major work specification. This solicitation process can reduce the total project schedule by months. It also helps to minimize risk to the prime contractor because the uncertainty of producing the material has been eliminated for that portion of the work.

6.10.5 Commonly available filter materials

In lieu of complete filter design, experience has shown that a modification to fine concrete aggregate as designated in ASTM C33 meets the design requirements for many foundation materials. This material is commonly referred to as "C33 concrete sand" or more simply "concrete sand." The additional requirement on the No. 200 sieve is needed to meet the permeability requirement of the filter design procedure. Table 6-3 gives the acceptable gradation band for this material. Because foundation conditions differ from site to site, this filter should always be checked against the gradation of the base soil (foundation soil).

Table 6-3. Modified gradation of C33 fine aggregate[1].

Sieve Size	Percent Passing by Weight
3/8-in.	100
No. 4	95-100
No. 8	80-100
No. 16	50-85
No. 30	25-60
No. 50	5-30
No. 100	0-10
No. 200[1]	0-2[2]

[1] Requirement beyond the ASTM C33 designation.
[2] Two percent stockpile, 5% in-place after compaction (see Section 7.4.3 later in this report).

In a similar manner, when modified C33 concrete sand is used as a filter, standard materials can be used as the gravel drain that surrounds the pipe. Several materials in ASTM D448 have been checked against modified C33 concrete sand and are included in Table 6-4. When using modified C33 concrete sand, the D448 materials do not have to be checked since the filter size is fixed. Three materials have been included since not all materials will be available in all areas.

Table 6-4. Gradation for ASTM D448 drain materials
(percent passing by weight).

Sieve Size	Blend 5791	No. 8	No. 89
2-in.	—	—	—
1-1/2-in.	100	—	—
1-in.	90-100	—	—
3/4-in.	75-85	—	—
1/2-in.	—	100	100
3/8-in.	45-60	85-100	90-100
No. 4	20-35	10-30	20-55
No. 8	5-15	0-10	5-30
No. 16	0-5	0-5	0-10
No. 50	—	—	0-5

[1] This gradation is a blend, in equal parts, of gradation Nos. 5, 7, and 9. It is not an ASTM standard aggregate.

Many state highway agencies also offer standard materials that may be acceptable in filter or drain applications. Each would have to be checked on an individual basis to assure that they meet the gradation design criteria. Also, aggregate suppliers may produce a material for another customer or application that meets the design criteria.

The use of "standard" materials from commercial sources can provide good economy over producing a "custom" gradation and a check of locally available "standard" materials should always be performed. Section 6.10.2 provides guidance on issues for consideration when performing this type of analysis. Navin (2006) produced a valuable tool for designing filters and drains including matching to commercially-available material mixes.

6.11 Recommendations

- Inclined chimneys should not be less than 5 ft wide (measured horizontally).
- Vertical chimneys should not be less than 3 ft wide (measured horizontally).

- Blankets should not be less than 18 in. thick (measured vertically) and should be place using a minimum of two lifts.
- When designing drainage elements on coarse foundations, the best understanding of foundation conditions will not be available until after excavation, so the design should be based on the worst conditions.

7 Construction

7.1 Introduction

Filter, transition, and drain zones must be constructed properly if a dam is to perform its function without incident or failure. The filter design process is not wholly completed until the filter construction is properly completed. Proper construction is essential to obtain a product with the properties assumed in the design of the embankment. Qualified field engineers are a vital piece of the design process because field adjustments and adaptations may be necessary. Some of the properties affected by construction include:

- *Density* – The overall density of the constructed zone depends on construction processes used. The uniformity of the density within the zone is also affected.
- *Gradation* – Placement of material at the specified gradation is essential for the filter/drain to perform as designed. Segregation and contamination of material are directly affected by construction procedures.
- *Geometry* –Proper construction is essential to obtain zones that are continuous at the width and geometry designed.

Proper construction procedures that can help ensure obtaining specified/ desired properties of filter/drainage zones include:

- Minimize segregation of sand and gravel particles
- Avoid contamination by other materials, particularly those in adjacent zones
- Maintain specified geometry (width and location)
- Maintain vertical continuity
- Achieve specified loose lift thickness
- Achieve desired or required% compaction or relative density in the case of an end-result specification
- Achieve a specified number of roller passes in the case of a method specification
- Ensure contacts with adjacent materials are adequately compacted

- Avoid using excess quantities of material caused by spreading of sand material during compaction
- Minimize particle breakdown

Methods and procedures to address these items and thus ensure the design intent is obtained during construction are discussed in this chapter. Much of this discussion is based on past experience in construction of filter/drains, transitions, and blanket drains in embankment structures (Hammer 2003).

7.2 Basic methods of construction of embankment filter zones

Three basic construction methods can be used to construct vertical and inclined sand filter/drains and transition zones in embankment dams (Hammer 2003):

1. Maintain the adjacent impervious core one lift ahead of the sand filter/drain
2. Maintain the sand filter/drain one lift ahead of the impervious core
3. Trenching

7.2.1 Maintain adjacent core one lift ahead of filter

While this method is not recommended for most applications, it is included for reference as a historical procedure. Steps utilized in this method of construction are shown in Figure 7-1. This technique has the advantage of minimizing spreading of sand material during compaction and could facilitate in obtaining the desired or specified% compaction or relative density. However, this method is more conducive to contamination of the sand filter/drain by adjacent materials falling into the section and from material being washed in during rains or by the spray from a passing water truck. Another disadvantage of this method is the difficulty in maintaining a specified filter width. Since adjacent materials are placed and compacted first (i.e., above the filter), there is a tendency for these materials to overlap into the sand filter/drain zone.

7.2.2 Maintain filter one lift ahead of core

The sequence of construction for this method is shown in Figure 7-2. This method has the advantage of inherently aiding in prevention of

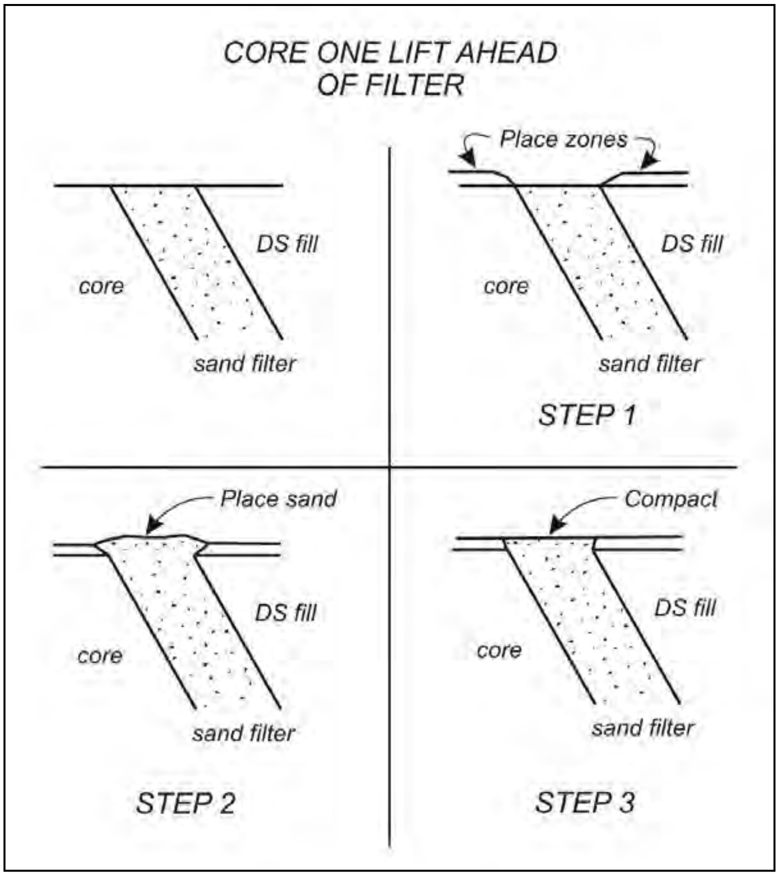

Figure 7-1. Steps in maintaining impervious core one lift ahead of a chimney.

contamination and in maintaining vertical continuity and full width of the filter/ drain. This is especially true if the embankment surface is maintained so that the filter/drain is the high point of the cross section, resulting in runoff and potential contaminants flowing away from the filter/drain zone. A disadvantage of this method is that compaction may be more difficult because the sand has a tendency to spread at its outer edges when compacted. Spreading also may result in a greater quantity of filter/ drain material being used in order to construct the required width. This could result in a significant increase in cost as the filter/drain is often the most expensive material in the embankment. However, experience has shown that these disadvantages may be significantly overcome by blading up a windrow of loose material at the edge(s) of the filter/drain as shown in Figures 7-2 and 7-3. The windrow should be of sufficient width to effectively contain the filter/drain material, thereby minimizing spreading during compaction. Although this method may result in using additional drain material due to a small "Christmas tree" effect, the extra cost is a

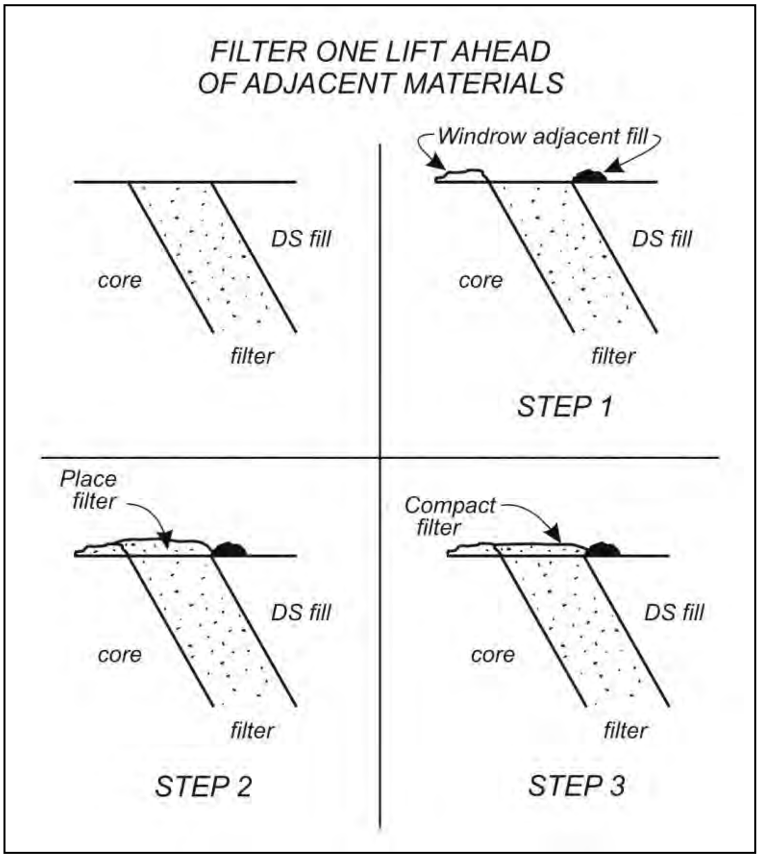

Figure 7-2. Steps in maintaining a chimney one lift ahead of impervious core.

small price to pay for ensuring that the drain width and gradation are constructed as designed. This method is especially applicable to filters/drains having a relatively narrow width.

7.2.3 Trenching

The trenching method is shown in Figures 7-4, 7-5, and 7-6 and is utilized when the filter/drain is constructed within a basically homogeneous impervious core. In this method, the impervious core is built completely over the filter/drain for a thickness of 3 to 5 ft. Using a trenching machine or other suitable excavation equipment, the core is then excavated down to the top of the previously completed filter/drain and the trench backfilled with compacted filter/drain material. The trenching method facilitates compaction since the material is confined on three sides, provides for closer control of quantities, and is conducive to obtaining excellent contacts between the filter/drain and adjoining impervious core. Disadvantages include the fact that trenching is time consuming, expensive, and

Figure 7-3. Windrowing impervious material adjacent to a filter/drain.

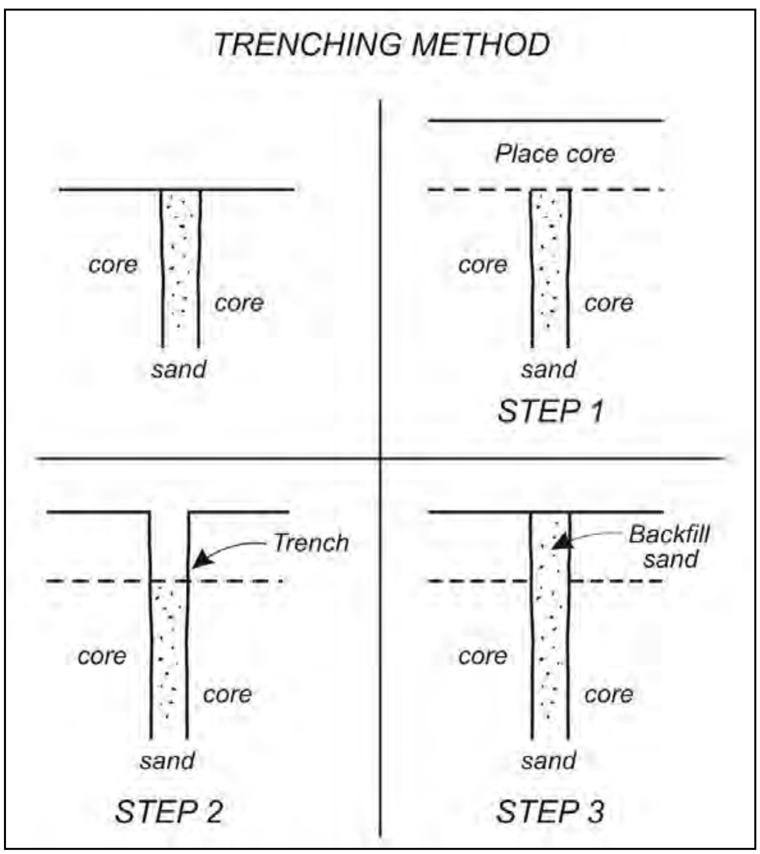

Figure 7-4. Steps for trenching method.

Figure 7-5. Trenching method – excavating trench.

Figure 7-6. Trenching method – backfilling trench.

inspection intensive (to ensure the tie-in between the existing filter/drain material and the newly placed material is not contaminated). In addition, this method can be used only for construction of narrow, vertical filter/drains in embankments composed of central and downstream homogeneous material that will stand vertically without caving when trenched.

7.2.4 Recommendations

Maintaining the filter/drain one lift ahead of adjacent zones is the preferred construction method and is recommended for most applications. Advantages inherent in this method, particularly with respect to contamination, definitely outweigh any extra effort required and possible increase in material quantities.

7.3 Specification items for filter and transition zone construction

Four primary items are generally specified in contract documents that relate to construction of filter and transition zones:

1. In-place material gradation, including material quality, durability, and angularity specs
2. Moisture (wetting requirements)
3. Compaction effort for a method specification or% compaction/relative density for an end-result specification
4. Geometry (alignment, width, and vertical continuity)

All specified requirements for these four items must be met during construction in order for the filter to function as intended by design. Additional discussion is presented later in this chapter.

7.4 Construction procedures

7.4.1 Importance

Because chimney and transition zone material quantities are relatively small compared to quantities of other embankment materials, and because construction of adjacent zones normally cannot exceed the top elevation of the chimney, construction of filter/drain zones is often on the critical path for embankment construction. Placement procedures are also slower because more care must be exercised during construction of filters/drains and transition zones. Because of this, there is often pressure to speed up construction of chimneys. However, cutting corners or deviating from established construction practices or contract specifications must not be allowed regardless of schedule pressure. Filter and drain zone construction should lead the embankment fill placement to reduce quality control problems and production efficiency losses.

7.4.2 Segregation

Completely eliminating segregation during construction is practically impossible since the material must be handled, and handling itself will cause some amount of segregation. However, adhering to proper construction practices that have been established by experience for storing, hauling, dumping, spreading, and compacting filter/drain materials can significantly reduce the amount of segregation (Navin 2006).

7.4.3 Particle breakage

All granular materials experience breakage during placement and compaction operations. Typically, loaders, and possibly dozers, place the material in stockpiles from which it is loaded into trucks, dumped onto the fill, bladed to a uniform loose lift thickness, and compacted by a smooth-drum roller. Each of these operations can cause individual particles to breakdown. This breakage is aggravated in crushed aggregates. This breakage leads to a change in gradation between what is produced at the plant and what is in place in the embankment. The Bureau of Reclamation (Reclamation) has been monitoring breakage between the source and the in-place fill on construction projects for the past 30 years by performing gradations at both points. Results of these gradations indicate particle breakage typically results in an increase in fines of between 1 and 3%, with 2% being typical for materials in the Western United States. It should be noted that generally source material from a crushing operation will experience greater breakdown than processed pit-run. Based on these data, gradations produced at the source should be 2% less than that desired in the embankment. When specifying material gradations at the processing plant, particle breakage should be taken into account and gradation tests run on in-place material. When material gradation is specified only for the fill, it will be the contractor's responsibility to address breakage between the plant and the fill. This situation can lead to delays and possibly claims by an inexperienced contractor.

For small projects, it may not be practical to determine aggregate quality by laboratory testing. In this instance, the designer should consider the mineralogy of the parent material. Quartz-based aggregates have higher quality than aggregates that come from sedimentary rocks. For materials obtained from commercial sources, stockpiles should be examined for slope uniformity. Piles with irregular or near-vertical slopes may indicate high fines content or possibly the presence of binders or cementing agents in the material.

7.4.4 Basic construction procedures

Basic construction procedures commonly employed to construct sand filters/drains are:

- Horizontal and vertical control
- Storage (stockpiling) of materials
- Loading of hauling equipment
- Hauling and dumping
- Spreading to specified loose lift thickness
- Wetting
- Compaction
- On-site and laboratory testing

Each procedure will be discussed in the following paragraphs with respect to maintaining the specified gradation (i.e., by preventing material segregation and/or contamination), the addition of water (if any), and attaining specified compaction effort or% compaction or relative density. In addition, the importance of maintaining 3-dimensional control of filter/drain geometry is discussed and recommendations presented.

7.5 Manufacture and storage

Whether materials for sand filters, drains, and transition zones are purchased off site or manufactured onsite, some type of storage, usually stockpiling, is necessary to ensure sufficient material is always available for construction. Stockpiling is most commonly performed in conical piles, which inherently cause segregation, whether by dumping from trucks or by belt discharge (Figure 7-7). Use of conical stockpiles should be kept to a minimum and carefully planned when used. Radial or tent-shaped stockpiles are preferred. Unfortunately, because little attention is usually paid to the stockpiling effort, a segregated material being loaded for placement in the embankment may result. More in-depth discussion of these problems, along with recommended mitigating actions and discussion of other types of stockpiles, is found in “Inspection and Sampling Procedures for Fine and Coarse Aggregate” (Indiana Department of Transportation, 2005). The following paragraphs present some additional causes of

Figure 7-7. Conical stockpile.

segregation that can occur during the stockpiling operation and methods to limit the amount of segregation that occurs.

7.5.1 Front-to-back segregation

Segregation begins on the conveyor belt where fines vibrate to the bottom and coarse particles remain on the top as the material bounces across the idlers (Figure 7-8). At the discharge point, if left un-deflected, coarse particles are thrown out and away while the fine particles tend to drop down and possibly under the discharge point. The greater the speed of the belt and drop height, the worse the particle segregation. This is known as front-to-back segregation and can be lessened by slowing belt speed, minimizing drop height, and utilizing baffles. Other mechanical changes can be made to the conveying system that will also help prevent segregation. These alterations are discussed in “Inspection and Sampling Procedures for Fine and Coarse Aggregate” (Indiana Department of Transportation 2005).

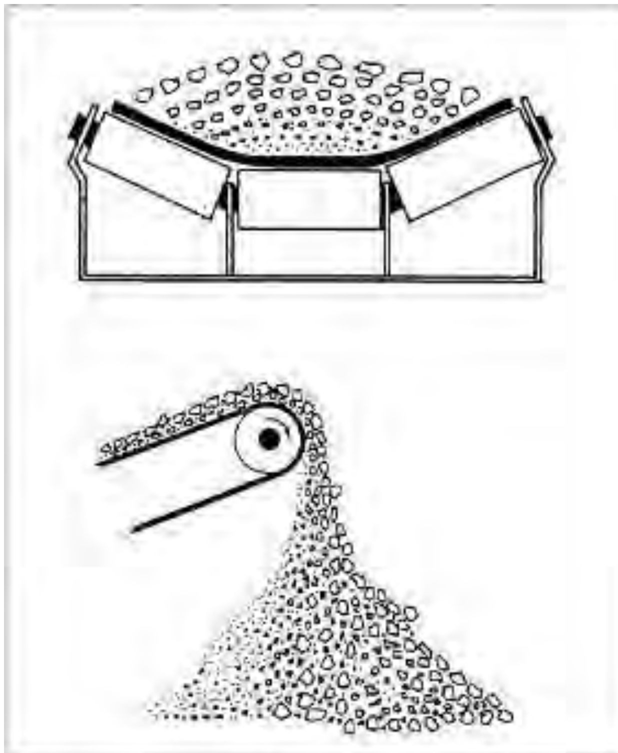

Figure 7-8. Belt segregation.

7.5.2 Roll-down segregation

Another common type of segregation is caused by piling aggregate so high that the larger particles roll down the sloped side of the pile as shown in Figures 7-9 and 7-10. The higher the pile and greater the drop height, the worse the problem. Obviously, segregation from this operation can be significantly lessened by limiting drop and stockpile heights.

7.5.3 Local contamination

Stockpiled materials can become contaminated by airborne dust and drainage runoff, resulting in an increased amount of fines in the material. Dust abatement procedures should be used to prevent contamination of fines into the stockpiled material. Positive drainage should be maintained so that suspended sediment is not carried into the stockpile (Navin 2006). A stockpile pad should also be used to minimize contamination between the stockpile and ground surface. Stockpile pads can consist of concrete, geomembrane, or an over-excavation backfilled stockpile material.

Figure 7-9. Segregation at high cone pile.

Figure 7-10. High drop height at belt discharge.

7.5.4 Loading hauling equipment

Loading of material from stockpiles is just as important as building the stockpile properly. Usually some mixing of the stockpiled material must be done by the loader prior to loading. Once excavation and loading has begun, no new material should be added to the pile near the loading area. The loader should work its way into the pile and mix the outer edges (coarse particles) to the center where the finer particles are located and then load the mixed material. The loader should be careful not to load material near the bottom of the stockpile because contamination and segregation is likely to occur. Rubber-tired loading equipment is preferred to tracked equipment since it reduces the amount of particle breakage. Just as in constructing stockpiles, drop heights from the loader into hauling equipment should be minimized at all times. Trucks should be loaded with a fairly flat top surface (as opposed to conical) in order to avoid segregation from vibration during transport. If dust conditions are severe, loaded material could be covered for transport to avoid contamination from dust. A more detailed treatment of loading procedures is found in "Inspection and Sampling Procedures for Fine and Coarse Aggregate" (Indiana Department of Transportation 2005).

7.6 Hauling and dumping

The hauling and dumping process also can contribute to segregation and contamination of filter/drain material if not carefully monitored to ensure that methods employed are those that will minimize detrimental effects. Normally, trucks are used for hauling filter/drain materials. On large jobs, either off-the-road large-end dump trucks (sometimes referred to as quarry trucks), as shown in Figure 7-11, or articulated trucks that can be end-, side- or bottom-dump are used. An articulated bottom-dump used to deliver filter and drain materials is shown in Figure 7-12. On smaller jobs, over-the-road end-dump trucks may be employed. Regardless of the type used, because of the possibility of contamination, tracking of hauling equipment on the filter/drain either must be prohibited or, if unavoidable, kept to the minimum necessary. If traversing the filter/drain material cannot be avoided during the dumping process, operators should be instructed that once they are on the filter/drain material, they should stay on because moving off and on again can increase the chance of transporting adjacent materials to the filter/drain, thereby causing contamination. Insofar as practical, material should be dumped as close to the required

Figure 7-11. Large end-dump truck utilizing an equipment crossing over a chimney filter.

Figure 7-12. Articulated bottom–dump truck. The photo illustrates difficulty that can arise when the truck dumps too quickly for the speed of the truck. The trailer will then hang up and require assistance from other equipment.

loose lift thickness as possible. Authorized crossing points should be established for all construction equipment (including pick-up trucks) that must cross the filter/drain. If bottomdump equipment is used and zone width allows, trucks should straddle the filter/drain material for discharge and use authorized crossing points for entrance and exit. Side-dumps are good for dumping filter/drain materials because they normally do not have to traverse the filter/drain. End-dump trucks are the most commonly used type of hauling equipment and should dump perpendicular to the longitudinal axis of the filter/drain in order to minimize tracking. This may require extra positioning in order to avoid dumping the entire load in one place (which often will require additional blading to properly spread the material). If the wheel base is wide enough, trucks should straddle the filter/drain for dumping. Equipment used for transport of filter/drain material may be earmarked for that purpose exclusively and not be used for other work. Truck boxes (beds) should be inspected regularly as the work proceeds because pockets of fine materials have a tendency to become concentrated in corners and may be released during dumping. All filter/drain areas traversed by equipment must be inspected and any deleterious material deposited from the treads removed. This requires constant attention and often may require hand work.

7.7 Spreading

7.7.1 By blading

Since spreading dumped material by blading inherently causes segregation and possibly contamination, blading should be kept to a minimum. Blading is usually accomplished by graders or dozers, as shown in Figure 7-13, with tracking off the filter kept to a minimum in order to lessen the chances of contamination. To minimize segregation, spreading equipment should be operated at minimum speeds and tracking on the filter minimized. Some hand work may be required in addition to blading.

7.7.2 By spreader box

One device for spreading sand filter/drain material that has been used on several jobs is a spreader box, as shown in Figure 7-14. Material is dumped into the spreader box bin, which is then pulled or pushed (depending on the particular operation) along the axis of the filter/drain zone. As the spreader box moves, material feeds out the rear of the box, releasing

Figure 7-13. Spreading sand filter material.

Figure 7-14. Basic single-bin spreader box.

material at the specified loose lift thickness and width. Use of this device can be somewhat cumbersome, but is usually worth the extra effort as no blading or trafficking by equipment (other than by the prime mover for the box) is required to place the filter/drain material in the exact loose lift thickness and zone width. Several variations of spreader boxes have been used, each being constructed to fit specific project requirements. At another project, the box was configured with a divider wall to place both fine and coarse filters/drains simultaneously, as shown in Figure 7-15. Each zone was 4 ft wide and was placed in a 12-in. loose thickness. The box was filled with material from either side, as shown in Figure 7-16, and towed by a Caterillar D-6 dozer at a slow speed, as shown in Figure 7-17. The spreader box shown in Figure 7-18 placed two 5-ft-wide zones simultaneously, but was fitted on the front of a dozer with hydraulic lift capabilities (both materials flowed out of the box at the proper zone width and loose lift thickness as the dozer operated in reverse as shown in Figure 7-19. Mobility of this type spreader box is significantly increased over that of a towed unit.

7.7.3 By truck-mounted conveyor

Beginning in 2000, contractors began using trucks outfitted with a conveyor for use in the placement of narrow width filters and difficult to access site conditions. The trucks, originally intended to deliver grain for agricultural applications, were modified to handle granular soils. The truck consists of a large box or hopper similar to a dump truck that holds the material and a conveyor mounted to the rear of the vehicle. The conveyor can both swing relative to the long axis of the truck and can be raised and lowered. This mobility is similar to that seen for the chute in concrete delivery trucks. Some trucks can be remotely operated using controls at the rear of the vehicle, allowing the driver to place material without assistance. Material is then delivered to the zone in a fashion similar to concrete placement. When care is taken, the material can be uniformly placed to the desired lift thickness, and no leveling is required. When less skilled operators are used, some raking by hand may be required, but spreading by a dozer is seldom needed. Figure 7-20 illustrates this operation in the construction of a 4-ft-wide chimney filter being added to an existing dam.

Figure 7-15. Double-bin spreader box.

Figure 7-16. Dumping into spreader box.

Figure 7-17. Towing spreader box.

Figure 7-18. Double-bin spreader box fitted to dozer – side view.

Figure 7-19. Double-bin spreader box fitted to dozer - front view.

7.8 Moisture (wetting) requirements

Experience has shown that because of the free-draining characteristics of granular materials, saturation to provide maximum density is very difficult to obtain/maintain and is no longer recommended. One way that has been attempted in the past to help accomplish saturation is to attach a spray bar to the roller so that the water is applied just ahead of the roller. A second option is to operate a water truck along with the roller so that the water may be applied manually just ahead of the roller.

However, both of these methods are time consuming, difficult to coordinate, expensive, and yield questionable results. Merely sprinkling the material prior to compaction was believed by many in the profession to be detrimental because this allowed bulking to setup in the material, which works against achieving maximum density. Fortunately, the capillary forces that develop during bulking can be easily overcome by dynamic compaction. It is now recognized that using vibratory compaction is the most effective way of densifying granular materials regardless of their degree of saturation. However, in view of the fact that high densities in filter/drain materials are usually not as important as obtaining a high permeability and a self-healing material, just wetting the material prior to

Figure 7-20. Truck conveyor delivering filter sand for the addition of a 4-ft-wide chimney filter to an existing embankment. Note that the material is uniformly placed from the conveyor, and no leveling is required. Dynamic compaction is provided by the roller shown in the foreground.

compaction may be the best way to moisten the sand. In addition, Milligan (2002) and others feel that moistened sand tends to segregate significantly less during handling than dry sand. If this is the case, the sand should be wetted prior to handling and again prior to compaction if it has lost significant moisture.

7.9 Compaction

7.9.1 General considerations

Filter zones are usually compacted for one or more of the following reasons:

- So they will not settle excessively on wetting.
- So they will not liquefy when loaded dynamically.
- So that a design shear strength will be achieved.

- To aid in obtaining strain compatibility with adjacent zones in the dam.
- Particle retention criterion is based on compacted material.

These characteristics normally require a relatively high density. On the other hand, there are valid reasons why sand filters/drains and transition zones should not be compacted to an excessively high density. Very densely compacted sands can result in overly brittle zones that have less than desirable self-healing properties. Requiring a high shear strength and low compressibility always has the accompanying properties of a more brittle zone with a tendency to crack upon deformation and to arch in narrower zones (McCook 2005). In order to achieve high densities, several passes of a heavy vibratory roller is normally required. This has a tendency to increase the potential of particle breakage that can produce a thin layer of excessive fines at the lift surface, which can have the effect of reducing vertical permeability while at the same time reducing self-healing properties of the material. Milligan (2003) states: "Compaction of filters should be minimal. Excessive compaction, particularly of crushed rock, can lead to the creation of sufficient fines in the filter to make them susceptible to cracking." The bottom line is that designers should be aware of and consider all the tradeoffs involved during project design and development of construction specifications for filter/drain and transition zones.

7.9.2 Types of compaction specifications

Two basic types of specifications may be used for requiring how materials are to be compacted: (1) end-result (*or performance*) and (2) method (*or procedural*). Each has advantages and disadvantages as discussed below.

End-result specification

The end-result type of compaction specification requires the filter/drain zone to be placed to either (1) a required% compaction or (2) a required relative density. The contractor determines what equipment and mode of operation are needed to accomplish the requirement. Other requirements are often included in the end-result type of specification, such as maximum permissible loose lift thickness. One example of this type of specification is to require in place material for a filter/drain to have a dry density that corresponds to at least 70% of relative density as determined by ASTM D4253 and D4254.

Advantages of an end-result type specification include:

- A cheaper bid for the work may be obtained because the contractors are allowed to develop their own method for achieving the specification requirement.
- Determining specification compliance is relatively straightforward. To check compliance, the density of the compacted fill is measured and then compared to the reference density test, usually minimum and maximum index density tests.
- Designers may have a greater confidence that the compacted materials have properties similar to those assumed in the design because the compacted densities are measured rather than estimated.

Disadvantages of the end-result specification are:

- To be most efficient, the contractor will use the minimal equipment size and number of passes that achieve the requirement. Because of working "on the edge," a greater chance occurs that the product achieved will not meet specifications. This can make specification compliance more important and more difficult for the owner.
- Judging work as it is being accomplished is always preferable to judging it after completion.
- Density test results are highly dependent on the location where the test is taken and on performance of the test itself. If test locations are not carefully selected to represent the overall compaction process, the results will not be representative.
- Density testing equipment and experienced inspectors are required, which adds to the cost of the method. Equipment is needed both to measure the density of the compacted zone in the field and to perform reference density tests. Relying on pre-construction testing of similar filters is not advisable because sand delivered to the site can vary from that tested prior to construction even though the sources are the same.

Method specification

This type of specification is termed "method" because it specifies to the contractor a method or procedure that is required rather than the result to be obtained. Acceptable equipment size and speed are usually specified along with number of passes and maximum loose lift thickness. These methods are based on the experience of the designer who is confident that the procedure will attain the desired results based on experience or prior work. A method-type specification is commonly used on larger projects in which considerable quantities of material are involved. These size jobs can also justify the use of test sections in which a particular method can be used to demonstrate that the desired density has been achieved. Experience has demonstrated the method specification to be more suitable for ensuring a quality project is attained, especially when large quantities of material are involved.

Advantages of a method type of specification include:

- Determination of specification compliance requires only verifying that equipment used was the type and size specified, that the material was spread to required loose lift thickness, and that the lift was compacted with the specified number of passes.
- Specification compliance requires no field or laboratory testing.
- Correction of a suspected deficiency while the work is being performed is more efficient than waiting for test results and correcting afterward. For instance, if the inspector thinks an area needs additional compaction, he can call for additional passes at the time the material is being compacted. If the contractor feels this work is beyond the scope of the contract, he may be compensated if a bid item entitled “additional roller hours” is in the contract. This way, the potential for disputes is lessened.
- This method requires continuous inspection by quality control personnel, which is an advantage over relying on testing alone.

Disadvantages of the method type specification are:

- Even though field and laboratory density testing are not required for contract compliance, such testing is required for verification that the method specified is resulting in the design unit weights and for as-built documentation.
- Continuous intensive field inspection by experienced personnel is required.
- Costs for inspection and testing may be higher than for the end-result type of specification.

7.9.3 Field compaction

The most effective types of equipment for compacting clean granular materials are those that employ vibration such as vibratory rollers or vibratory plate compactors. Vibratory rollers have a long and successful history in compacting clean sands like those used in filter/drain zones. D'Appolonia, et al. (1969) reported good compaction for ASTM C33 clean sand using a relatively small (12,500-lb) roller. That research concluded that compacting relatively thin clean sand lifts with at least two passes of a lightweight vibratory roller obtained good compaction results, which equaled about 75% relative density for the sand evaluated. Wetting sand prior to compaction is typically recommended but may not be required; on-site testing should determine necessity. Dynamic loading by the compactor is the critical component in compacting granular materials and should always be used regardless of the equipment size. When vibratory compaction is used, the water content of the material is not as critical.

Vibratory compactors

Vibratory compactors or "rollers" range in size from large double-drum types to smaller "walk-behind" drum or plate models. Examples of these type rollers are shown in Figures 7-21 through 7-23. Specifications normally require one or more of the following characteristics when "method specifications" are used in a contract:

- Static weight
- Drum diameter and width
- Range of operational frequencies of vibration
- Imparted dynamic force
- Roller operation (covered in following section)

All specified static and dynamic properties of the particular roller must be checked and verified as being in accordance with the specification requirements prior to use.

Compactor operation

Operation of the approved roller will be specified in terms of number of passes, overlap between passes, maximum speed of operation, and operating frequency. In addition, there may be additional operating

Figure 7-21. Double-drum vibratory roller.

Figure 7-22. Single-drum vibratory roller.

Figure 7-23. Walk-behind vibratory plate compactor.

requirements relating to turning and backing. The roller must be in motion when the dynamic force is engaged or disengaged. Also, the roller should not be permitted to sit idle with the dynamics engaged since this will lead to "digging-in" and over-densification of the filter. The number of roller passes on each lift, as well as roller overlap (usually a minimum of 1 ft), must be verified by field observation. A roller pass of a smooth-drum vibratory roller is defined as a complete coverage of the area to be compacted with each trip of the roller. One pass of a double-drum roller is normally equivalent to two passes of a single-drum roller. Since these terms are subject to interpretation, these definitions should always be included in the specification. Roller speed can be readily checked by timing the movement of the roller over a known distance until the inspector is comfortable in visually assessing the speed.

Compaction of contacts with adjacent materials

Contacts between the filter/drain and adjacent materials, such as between the filter/drain and the impervious core, must be adequately compacted. If left uncompacted, an area of low shear strength and high compressibility could develop along the contact. Compaction of zonal contacts can be overlooked rather easily since the filter/drain is compacted by smooth-drum vibratory rollers and the impervious core is normally compacted by a tamping (sheepsfoot) or a rubber-tired (pneumatic) type roller. Equipment operators of each type of roller are often given instructions to avoid tracking on adjacent zones. Each operator working in accordance with his instructions may result in the area around the contacts not receiving adequate compaction.

Proper compaction of the contacts is accomplished by overlapping the vibratory roller onto the adjacent material rather than overlapping the tamping roller onto the filter/drain. However, roller operators and inspectors should be taught that a minor amount of mixing of the two adjacent materials is less a detriment than leaving the contact uncompacted. An overlap of 1 ft is usually specified. To facilitate compaction of contacts, all grade stakes used to mark zonal contacts prior to compaction should be removed so that operators do not drive around the stakes. Density testing should be conducted at or near zonal boundaries to verify that adequate compaction is being achieved in these critical areas. An example of rolling a sand filter/drain contact is shown in Figure 7-24 (Hammer 2003).

7.9.4 Recommendations

The following recommendations are made concerning moisture and compaction of filter and transition zone materials:

- Filter materials should be wetted prior to handling to facilitate handling as well as to help minimize segregation.
- The designer should consider all implications when specifying a desired density for the filter/drain material. A relative density of 70% is often used as a criterion value for minimum acceptable density.
- A method-type specification is usually recommended.
- Compaction of filter materials should be by means of vibratory rollers with the minimum required effort specified that will attain the desired density.

Figure 7-24. Compacting a joint between two zones by a vibratory roller.

- Avoid over-compaction, which can lead to particle breakage.

7.10 Horizontal and vertical control

Horizontal and vertical control of sand filter/drain and transition zones in an embankment is vitally important during construction. See section 6.4 for additional explanation of errors, such as the "Christmas tree" effect, that can be introduced by poor survey control as it relates to chimneys. Each lift of these zones must be accurately surveyed and staked or otherwise marked with temporary control points to ensure proper geometry is maintained at all times. Surface locations of previously placed lifts should not be used to establish the location of subsequent lifts due to possible errors resulting from over or under-building, spreading, or surface unevenness. To facilitate proper horizontal and vertical control, the surface of the filter/drain (and the entire embankment for that matter) must be maintained at the same longitudinal elevation. Every effort must be made to accomplish this during construction, even to the extent of building partial sections of the filter/drain in low areas in order to ensure a level surface is maintained. It should be noted that this type of control is applicable to construction on an embankment surface. Other areas that may be more confined, such as adjacent to conduits and pipes, must have their own type of control that fits the particular situation at hand.

7.11 Protection of completed work

7.11.1 Embankment surface during construction

Surface grade

The surface grade of the embankment should be maintained in such a manner that the filter/drain is protected at all times from contamination by surface runoff (Figure 7-25). To accomplish this, the filter/drain should be maintained at the crown of the embankment surface and protected by whatever means necessary (grading, windrows, etc.) at the end of a shift or when impending storms are forecast. In addition, the embankment surface should not contain low areas, especially those that involve filter/drain zones. Inspectors should watch for contamination resulting from over-zealous water truck operators on adjacent zones. Whenever contamination of filter/drains occurs, all contaminated material must be removed prior to resumption of normal placement operations.

Figure 7-25. Surface water contamination of a chimney filter.

Haul road crossings

In order to construct a zoned embankment, equipment used to construct other zones must inevitably cross the filter/drain/transition zone (Figure 7-26). Equipment crossings are fraught with potential for contamination of the filter/drain, for reduction in filter/drain width, and for the

Figure 7-26. Haul road crossing of a chimney filter and drain.

filter/drain to be partially or completely cut off vertically. Therefore, special measures must be taken to ensure that the crossings do not adversely affect the design cross section or the desired properties of the filter/drain. Equipment crossings must be controlled; they must be kept to the absolute minimum necessary and must be in definite and confined locations. All personnel working on the dam must be instructed as to crossing locations and the importance of utilizing them.

Assuming crossings are kept to a minimum and are at specified, confined, and well-marked locations, one method to protect the filter/drain is to place a "sacrificial pad" of drain material at each crossing. This pad should be wide enough to accommodate equipment being used and should have a minimum thickness of 18 in.. When the crossing is no longer needed, the pad and drain material below the crossing and well beyond its width are excavated and the drain brought back to desired grade with clean, well-compacted filter/drain material. Excavation of a crossing is shown in Figure 7-27. Another method requires the placement of a heavy geomembrane or steel plates over the drain at the crossing to help protect the material from effects of vehicle traffic. Placement of a geomembrane is shown in Figure 7-28. Even with the use of a geomembrane or steel plates, some undercutting and backfilling of the filter/drain material will still be required, but usually not to the extent required without the covering.

Figure 7-27. Excavation of filter material under equipment crossing.

Regardless of the method of protection used, a passing gradation test should be performed on in-place material prior to allowing placement of additional filter/drain material. Such a test would provide verifiable assurance of the site's condition as well as a documented record of acceptable crossing cleanup practice.

Embankment surface during winter shutdown

Experience indicates that providing protective covering for an embankment surface during winter shutdown is not necessary in most parts of the country. In areas where frost penetration is expected, a loose cover several

Figure 7-28. Placement of geomembrane at crossing over a chimney filter and drain.

feet thick should be placed. When construction re-starts, this protective layer is removed (Sherard et al. 1963). The worst damage that occurs in frozen material is a moderate loosening of the upper few inches of the completed embankment due to frost action. If this loose surface layer is found, it should be excavated in the spring before the next lift is placed. The depth of stripping required can be best determined by visual evaluation of the upper portion of the embankment using shallow test pits. The key to any embankment protection scheme for winter shutdowns is to ensure that the material upon which the first lift is placed in the spring is in full accordance with the specifications of all required properties.

7.11.2 Damage to pipes

Proper methods for installing plastic pipe are described in FEMA P-676. Horizontal drains are often utilized to collect seepage, typically in toe drains. A number of poor practices are commonly encountered in pipe installation and should be avoided. They include, but are not limited to:

- Compaction of backfill using the backhoe bucket by “thumping” or setting the bucket on the backfill and lifting the back of the backhoe by applying pressure to the bucket
- Wheel rolling, either parallel or transverse, to the pipe by any type of construction equipment or vehicle
- Not placing or fully compacting backfill under haunches of the pipe
- Haul roads or equipment crossing the pipe without sufficient cover.

A minimum depth of 4 ft should be provided over the top of the pipe for H-20 highway truck loading (front axle load of 8,000 lb and rear axle load of 52,000 lb) in accordance with American Association of State Highway Transportation Officials (more depth may be required if recommended by the manufacturer). Note that crossing over a pipe at a low point in the haul road will lead to higher-than-normal loads due to braking. In a similar fashion, a poorly maintained and uneven haul road will lead to bouncing, which also results in higher loads. If the haul road is poorly maintained or does not have a uniform grade, traffic should be restricted to no more than 5 mph at the crossing. Recommendations for the type of pipe to use for these loading conditions are presented in section 2.4.2. To confirm that installed pipes have not been damaged, it is recommended that a video inspection be made soon after 4 ft of fill has been placed over the pipe.

7.12 Ensuring a quality product

Proper control during construction must be treated as an important and integral part of the process by which a project grows from inception to reality. The importance of effective control of construction cannot be overemphasized. No matter how thorough and complete a design may be, without proper control during construction, there cannot be any great degree of confidence that the desired end product has been attained. Not only must the construction be in accordance with contract documents (plans and specifications), but thorough documentation also must exist to demonstrate that such is the case. The system by which this control is implemented consists of two activities: quality control (QC) and quality assurance (QA).

7.12.1 Quality control

Quality control is generally defined as a planned system of activities or processes that ensure the intended product quality is attained. Specifically for construction of filter/drain/transition zones in embankments, the purpose of QC is to ensure the specification requirements are met and to provide as-built documentation of the constructed product. Normally, QC will consist of field inspections, field measurements, surveying, and laboratory testing. Quality control may be exercised by the contractor or by the owner. Based on experience with larger projects, QC is most effectively administered by the owner, independent of the contractor. Regardless of who is responsible for implementation, a QC program cannot be effectively carried out without a full staff of experienced inspectors and laboratory personnel. This is relatively expensive, but is just as much a part of project cost as is design. The old adage that states "quality costs least" is certainly true in the case of embankment dam construction and particularly construction of filter/drain and transition zones. The contractor will be performing construction activities as fast as possible, and schedules and milestones must be met in accordance with the contract documents.

Although time is of secondary importance to quality, QC staff must be capable of performing their work in a timely manner that does not slow down construction unless problems are encountered.

7.12.2 Quality assurance

Quality assurance is considered an oversight program of QC to ensure that verification and documentation procedures of the QC program are being accomplished correctly and to provide additional data for documentation. As such, QA will consist of essentially the same items as QC. The owner should have, and should administer, the QA program, whether with in-house personnel or with contract personnel.

7.12.3 Design of QC/QA programs

Because of intimate knowledge of the design and its intent, the project designer should have major input into development of QC/QA programs. The designer knows best which items must be monitored and at what intensity and frequency. If the contractor is to perform the QC program, the designer should review and approve the plan. If the owner is to perform the QC program, the project designer should design the plan with input from field personnel who will be responsible for contract administration. The designer should develop the QA program in either case since the QA program is administered by the owner.

7.12.4 Documentation

Maintaining detailed documentation of QC/QA results is imperative since these results are not only used for contract compliance but also provide vital information for documentation of the as-built condition of the structure. This information should be summarized in a report at the completion of construction and filed in a readily accessible manner in the event questions or problems arise during operation of the structure.

7.12.5 Communication

Experience has clearly shown the importance of establishing and maintaining good communication between all parties involved in the design and construction process. Prior to the beginning of construction, the designer(s) should communicate to all construction personnel the intent of the specifications (which may not be clear from reading the specifications themselves). Complicated designs are especially in need of explanation. To ensure such communication takes place, the U.S. Army Corps of Engineers (USACE) requires that the designer prepare a document entitled “Engineering Considerations and Instructions for Field Personnel.” This document not only explains the design intent but also presents requirements for testing and inspection to ensure specification compliance and that the project is properly documented. To further enhance communications, most USACE offices also require that this document be presented to field personnel at an onsite meeting. Also, since fewer and fewer dams are being constructed, there are fewer contractors with experience in embankment construction. Often, a contractor is selected to construct an embankment dam whose primary experience is in highway construction. Thus, there may be a learning curve for the contractor that can be significantly reduced by a similar type of communication between field personnel and the contractor early in the construction process. Regardless of the size of a project, good communication will go a long way toward keeping problems to a minimum.

7.13 Inspection

"There are few things of more importance in ensuring quality on a construction job than to have a set of eyes attached to a calibrated brain observing the construction operations" (Peck 1973). Regardless of the number of tests performed, they represent only a minute fraction of material that has been placed. Therefore, continuous inspection, or observations of field operations and conditions, is the backbone of the QC program and is vitally important to ensuring quality. Typical items an inspector should observe with respect to filter/drain/transition zone construction include the hauling, dumping, spreading and compaction operations; condition of the in-place material; and protection of completed work. In addition to observation, the inspector must call for testing to be performed at the locations he determines. All of these operations should be observed and monitored with respect to specification compliance and proper construction practice. Details of inspection of these operations have been discussed previously in this chapter.

Inspection personnel must be experienced, knowledgeable of the plans and specifications, and exhibit good communication skills. Early in the job, inspectors must make every effort to become "calibrated" to material characteristics and behavior in the construction process. This is necessary to lend credibility to the observations and to operate efficiently. By observing construction processes and material conditions, a properly calibrated inspector should be able to have a very good idea whether placed material meets the specifications even before testing is performed. Inspectors must communicate well, especially with contractor personnel. Experience has repeatedly shown that inspectors should get to know the people who do the work and explain to them why things such as removing contaminated material or maintaining specified minimum drain width are important. When properly motivated by knowing the reasons for and importance of the work they are performing, laborers and equipment operators usually will take more pride in their work, resulting in an additional, though informal, QC force. To be most effective, inspectors must establish a reputation for being strict but fair. Documentation of inspection operations is in the form of inspection reports, which are prepared daily by the inspector and reviewed by supervisory personnel.

7.14 Testing

Field and laboratory testing together provide verification of specification compliance for filter/drain and transition zone materials placed in an embankment. In addition, test results provide as-built documentation for the completed structure. Test results aid in the calibration process for QC personnel and serve as indicators for the contractor as to what is expected to be achieved in the field. Like inspection, field and laboratory testing form an integral part of the QC program and are essential to obtaining product quality. Because of time constraints, most projects will require an onsite testing laboratory staffed with trained and experienced technicians. All inspection and laboratory technicians should also be experienced with the latest testing procedures and requirements. All test and sample locations must be accurately surveyed and recorded as a matter of record.

7.14.1 Field testing

One aspect of field testing for construction of filter/drain and transition zones consists of in-situ tests to determine dry density. End-result specifications require measurements to be made of the compacted filter/drain zone to determine specification compliance and for documentation. Construction testing for specification compliance is not required under a method-type specification. However, even when using method specifications, field measurements of the compacted density of the zones should periodically be made to ensure that the method specified is achieving the desired results and to provide as-built documentation. Field density tests should be performed by either the nuclear method (ASTM D6938), the sand cone method (ASTM D1556), or other suitable testing method. A typical nuclear meter for density and water content determination is shown in Figure 7-29. The sand cone method may be difficult to perform in dry, clean granular materials due to caving of the sides of the excavated hole. The nuclear method is much easier to perform since it is a near surface test and requires much less time to perform and obtain results. Use of the method to determine field dry density and water content requires that the instrument be frequently calibrated to the particular material being tested. Most problems reported with the nuclear density test can be traced to improper calibration of the instrument. Because the surface of compacted sands is often “fluffy” due to lack of confinement, the nuclear test should be performed below the surface after carefully preparing a deeper undisturbed surface by blade and hand. Most organizations perform density testing on the approximate surface of the underlying lift. Care in preparing a proper surface and ensuring the intended location and depth of material is being tested is very important. The sand cone test is considered by both Reclamation and the USACE as the "gold standard" test for determination of in-situ dry density. On major water resource projects in both organizations, in-situ dry density may be determined using the nuclear meter, but not without frequently correlating to the sand cone test.

Figure 7-29. Typical nuclear moisture-density meter.

Reclamation testing data indicate that the nuclear meter frequently underestimates dry density (Reclamation, 2002).

Selection of test locations

The selection of field density test locations should be made by the inspector who has been observing construction operations. Factors that affect selection of test locations should be discussed with the technician performing the test. Improper selection often may cause more difficulty in practice than many of the errors in the test procedure itself. The test location should be selected with a view toward obtaining both the average percent compaction and the percent compaction in any area where the inspector suspects under or over-compaction has occurred. Over-compaction of sands could result in the development of fines at or near the lift surface, which could adversely affect the maximum percent fines requirement. Similarly, locations where samples for gradation verification testing are taken should also be selected by personnel who have observed placement and compaction operations. Gradation tests for specification compliance are performed on sand samples after compaction. As was the case with density tests, locations for gradation tests should be selected based on visually determining that the location selected is representative of the overall construction process. Selected locations may also be based on observations where the inspector suspects the specified gradation has not been met. All sampling locations must be accurately surveyed (xyz position) for test repeatability and records.

Frequency of testing

The frequency at which testing for density and gradation is performed should be established by the designer before construction starts. Test frequencies are normally based on a volume-placed basis, although increased testing may be required when the placement volume is relatively low but the height of fill placed is significant, such as in confined or concentrated areas. Table 7-1 shows an example of frequency requirements specified by the designer for construction of a large embankment dam. It is noted that the values shown represent minimum test frequencies and must be increased at the beginning of the project and when there are problems or other extenuating circumstances.

Table 7-1. Example of minimum testing frequency for filter and transition materials on a large project using a method specification for compaction.

	Type of Test	Number of Tests Zone 2 – Filter (sand)	Number of Tests Zone 3 – Transition (sand and gravel)
QC QC	Gradation Density	1 per 2,000 cu yd None required	1 per 5,000 cu yd None required
QA QA	Gradation Density	1 per 5,000 cu yd 1 per 5,000 cu yd	1 per 10,000 cu yd 1 per 7,500 cu yd

Note: The testing frequency shown is the minimum acceptable rate. More frequent testing may be required.

7.14.2 Laboratory testing

Reference density

Several types of control tests have been and are currently used to obtain reference density values for design and construction of granular filter zones. The primary types of tests used are:

- *Relative Density Test* – Minimum Index Density ASTM D4254, Maximum Index Density, ASTM D4253
- *Compaction Test* – ASTM D698 and ASTM D1557
- *Vibratory Hammer Test* – ASTM D7382.

The following sections discuss these tests for use in construction control in more detail.

Relative density

Minimum and maximum index density tests can be performed on a wide range of filter materials ranging from fine concrete sand to gravels as described in ASTM D4253 and D4254. After establishing the minimum index density and the maximum index density for the material, the in-place value is established that provides the basis for permeability and shear strength values. This in-place (intermediate) value is known as the relative density. Using this procedure, a minimum relative density of 70% has been frequently specified as the required density for granular materials. Using relative density to control the placement of granular filters has a long tradition, but problems with the test have caused designers to explore other methods for establishing design densities and writing specifications for placement. Problems with the relative density test include:

- Difficulty in calibrating the vibrating table used for the maximum index density test
- Poor repeatability of test – lack of precision
- Lack of equipment near construction site and cost of tests.

Tavenas, et al. (1973) and Holtz (1973) describe problems with the use of relative density in construction control. They report unacceptably large deviations in test results on a standard sample between laboratories. Their studies show that results from the minimum and maximum index density tests are subject to large variations even though standardized procedures were prescribed for the testing. Tests showed a 95% confidence interval for the minimum index density test. As an example, for clean sand, the spread was 6.8 lb per cu ft. In addition to the above problems with the test itself, performance of the test is time consuming and is therefore not conducive for compliance testing during construction.

Proctor maximum density

Determination of maximum density by what is commonly known as the "Proctor" or impact compaction test has been utilized for many decades. There are two basic types of this test, the difference being the amount of energy used to compact the soil. Since the Proctor test is used primarily for impervious soils where maximum density and optimum water content values are needed, it is rarely used for pervious soils and is not recommended. For example, typical moisture-density curves for clean sand are shown in Figure 7-30, which indicate that the maximum density for these types of materials occurs when the material is nearly dry or completely saturated. This fact has led to the development of correlative one-point tests, the results of which may be used as the reference maximum density. Use of these correlative methods is much faster and requires much less effort than performing the full compaction test and is not recommended for compliance testing of granular filters.

One-point (wet) compaction test

This procedure, developed by Poulos (1988), involves performance of a single compaction test by the modified compaction test method (ASTM D1557) to develop a correlative density value termed index unit weight.

This value then serves as a reference with which to compare the field density test result in order to obtain a value of percent compaction. The test is performed on wet sand (near saturation). Equipment required to perform this test is commonly available, and the procedure is familiar to most technicians. The test is recommended by Poulos for all sands, but particularly for clean sands having less than about 5% fines passing the No. 200 mesh sieve and high coefficients of permeability. This would be applicable to most filter sands.

One-point (dry) proctor test

In 1996, McCook developed correlations between a one-point standard energy compaction test (ASTM D698) on dry sands with test results obtained from performance of max-min relative density tests. Based on these results, the one-point compaction test on dry filter appears suitable for investigation as the design and specification basis for compaction of filter/drain materials and other sands. More detailed information on this

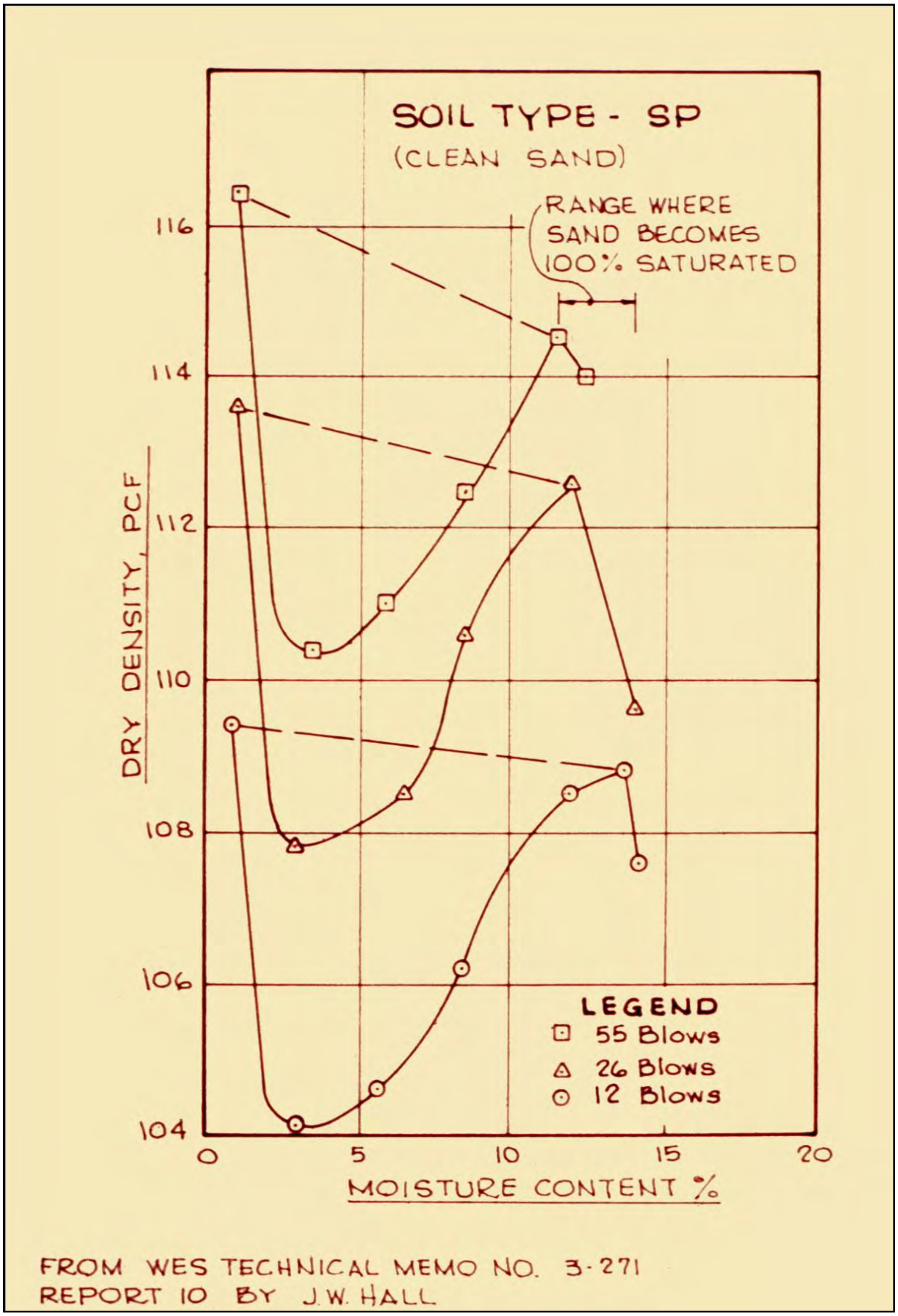

Figure 7-30. Typical compaction curves for a clean sand.

method is found in "Correlations Between a Simple Field Test and Relative Density Test Values" (McCook 1996).

Vibratory hammer compaction test

Another development in obtaining index density values for clean sands is a test using a vibratory hammer (Figure 7-31 shows the equipment). Prochaska (2004) and Drnevich et al. (2005) discuss the test in detail. The test is ASTM Test Standard, ASTM D7382. A reference density is obtained in the test by compacting a sample of filter into a steel mold with a hammer, shown in Figure 7-31, using three lifts to fill the mold. Either the filter is oven-dry or saturated during the test. Two sizes of mold are used. A 6-in.-diam mold is used for filters with a maximum particle size of 3/4 in., and an 11-in.-diam mold is used for filters with particles with a maximum

Figure 7-31. Vibratory hammer used to obtain a reference density value for filter materials. (Photo courtesy of Dr. Vincent Drnevich)

size up to 2 in. in diameter. The value obtained for the vibrated dry density is used as a reference density for laboratory tests and can be used in conract language to specify a minimum acceptable density for the material tested.

7.14.3 Gradation

Laboratory testing of sand samples for gradation compliance is accomplished by utilizing the test method presented in ASTM D422, "Standard Method for Particle Size Analysis of Soils." In cases where a quick check is useful (as perhaps percent passing the No. 200 sieve size), a partial gradation may be performed. Otherwise, the sieves used in the test should be the same size and number as presented in the specification.

7.14.4 Particle durability

As long as the approved source of material remains constant and particle breakage is not excessive, testing for particle durability during construction is usually performed at a low frequency for confirmation purposes. Since testing frequency is low and time is normally not critical, particle durability testing is usually performed at an offsite testing laboratory. Chapter 4 contains information on tests to evaluate durability of aggregate sources. As a minimum, the material should meet the durability requirements of concrete aggregate as defined in ASTM C33.

7.15 Application of test results

Results of all testing should be communicated to the inspector as soon as available. If test results indicate specification requirements are met (usually termed a passing test), then no further action is required except to digitize the data into the proper database. If test results indicate that specifications are not being met (termed a failing test), then action must be taken by an appropriate individual to communicate to the contractor that the problem needs to be corrected. All test results should be promptly digitized to facilitate quick retrieval and performance of various required data analyses.

7.15.1 Compaction requirements

In the case of a failing% compaction or relative density test, additional roller passes may be all that is required to correct the deficiency.

Re-testing after additional compaction should be performed. If re-testing determines the material is in accordance with the specifications, the failing test result is replaced by the passing test result for specification verification and as-built purposes. The failing test result is maintained for the record, but is not used for as-built documentation since it is no longer representative of in-place material.

If the material does not pass after additional passes, the equipment should be inspected for malfunction of the vibratory system. If it is not functioning as specified, it should be repaired or the machine replaced. When additional passes are made in situations like this, the gradation should be checked after density is achieved to assure that the fines limit has not been exceeded due to material breakdown.

7.15.2 Gradation requirements

In the case of a failing gradation test, the material represented by the test result must be removed and replaced with compacted material that meets the gradation requirements. Again, re-testing is required, and the passing test result should replace the failed test result for specification verification and as-built documentation. The failing test result should be kept for the record, but not used for any as-built purpose since the material it represented has been removed.

7.15.3 Recommendations

The use of a correlative method to determine laboratory reference values of density is recommended, and based on simplicity and equipment availability, the one-point test methods for obtaining reference maximum density values with which the field density test is compared are recommended. However, the method employed should be that which is most suitable to the site materials and yields the best correlations. Studies should be performed prior to selection of a particular method to ensure that the method is most applicable to the materials being utilized. The use of relative density testing or 4- to 5-point full density testing for compliance testing during construction of filter/drain and transition zones is not recommended.

References and Additional Reading

American Concrete Institute, Controlled Low Strength Materials (CLSM), ACI 229R-94. July 1994.

American Water Works Association, Concrete Pressure Pipe, AWWA M9. 1995.

Arulanandan, K. and E.B. Perry, "Erosion in Relation to Filter Design Criteria in Earth Dams," Journal of Geotechnical Engineering, ASCE, Vol. 109, No. 5, 682-698. 1983.

ASTM International, "Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density," Annual Book of ASTM Standards, D4254-00. 2000.

______, Designation C33-08, "Standard Specification for Concrete Aggregates," ASTM, PO Box C700, West Conshohocken, PA 19428. 2008a.

______, Designation D448, "Standard Classification for Sizes of Aggregate for Road and Bridge Construction," ASTM, PO Box C700, West Conshohocken, PA 19428. 2008b.

______, Standard Test Method D7382, "Standard Test Methods for Determination of Maximum Dry Unit Weight and Water Content Range for Effective Compaction of Granular Soils Using a Vibrating Hammer,." ASTM, PO Box C700, West Conshohocken, PA 19428. 2008c.

Babaeyan-Koopaei, Valentine and Ervine, "Case Study on Hydraulic Performance of Brent Reservoir Siphon Spillway," ASCE, Journal of Hydraulic Engineering, Vol. 128, No. 6, pp. 562-567. June 2002.

Bertram, G.E., An Experimental Investigation of Protective Filters, Harvard University, Soil Mechanics Series No. 7, Publication Number 267, 1940, Graduate School of Engineering, Publication No. 267. 1940.

Bolton, A., et al., "Chemical and Biological Causes of Cementation in Granular Soils," Virginia Tech, Blacksburg, VA, prepared for Reclamation. 2006.

Britton, Jeremy, "Internal Erosion and Piping," Power Point Presentation at Tri-Service Infrastructure Systems Conference & Exhibition, St. Louis, MO. 2-4 August 2005.

Building Research Establishment, Burenkova, V.V., Assessment of suffusion in non-cohesive and graded soils, Filters in Geotechnical and Hydraulic Engineering, Brauns, Heibaum & Schuler (eds.), Balkema, Rotterdam, pp. 357-360. 1993.

Bureau of Reclamation, "Laboratory Tests on Protective Filters for Hydraulic and Static Structures, Laboratory Report No. EM-132. 1947.

______, Influence of soil properties and construction methods on the performance of homogeneous earth dams, Technical Memorandum 645, Denver, CO. 1953.

______, "The Use of Laboratory Tests to Develop Design Criteria for Protective Filters," Earth Laboratory Report No. EM-425. June 20, 1955.

______, "Laboratory filter tests on concrete sand and gravel,." EM-588, Denver, CO. 1960.

______, Design of Small Dams, Second Edition. 1973.

______, Design of Small Dams, Third Edition. 1987a.

______, Design Standard No. 13, Embankment Dams, Chapter 8 – Seepage Analysis and Control. 1987b.

______, Guidelines for Controlling Seepage along Conduits through Embankments, ACER Technical Memo No.9. 1987c.

______, Analyses of Geological and Geotechnical Issues at Lake Alice Dams No.1 and 1-1/2 for Modification Decision Analysis, Tech. Memo No. LA-3620-MDA-1, (Internal Report). 1990.

______, Characteristics and Problems of Dispersive Clay Soils, by Paul Knodel, Technical Service Center, Denver, CO. October 1991.

______, Swihart, Jay, "Full Scale Laboratory Testing of a Toe Drain with a Geotextile Sock," DSO-98-014, Technical Service Center, Materials Engineering and Research Laboratory, Denver, CO. December 1997a.

______, Swihart, Jay, "Laboratory Pipe Box Test on Toe Drains at Lake Alice Dam and Enders Dam," Technical Service Center, Materials Engineering and Research Laboratory, Denver, CO. December 1997b.

______, "Earth Manual – Part 1, 3rd edition," Earth Science and Research Laboratory Geotechnical Research, Denver, CO. 1998.

______, Project Data Files. 2002.

______, "Investigation of the Hole Erosion Test used to Determine Erosion Indices of Soils for Evaluating Risk of Internal Erosion in Embankment Dams." August 2004.

______, "Design Standards No. 13: Embankment Dams," Chapter 5 – Protective Filters, Technical Service Center, Denver, CO. 2007.

______, "Technical Report of Findings – Risk Analysis of Potential Interim Corrective Measures – AV Watkins Dam, TM No. AW-8313-2, Appendix 1, Department of the Interior, Bureau of Reclamation, Technical Service Center, Denver, CO. April 2007.

______,Snortland, Nate, "Petrographic Examination of Exhumed Filter Sand-Horsetooth Dam Modifications." November 2007.

______, "Physical Properties of Plastic Pipe Used in Reclamation Toe Drains," Report DSO-09-01, Dam Safety Technology Development Program, Denver, CO. September 2009.

Carrier, W. David III, "Goodby, Hazen; Hello, Kozeny-Carman," Journal of Geotechnical Engineering, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 129, No. 11, pp. 1054–1056. November 1, 2003.

Casagrande, Arthur, Discussion in Main Session 3, page 305, Seventh International Conference on Soil Mechanics and Foundation Engineering held in Mexico City in 1969.

Cedergren, H.R., "Seepage Requirements of Filters and Pervious Bases," ASCE Paper No. 2623, Transactions, Vol. 127, part I, pp. 1090-1 1 13. 1962.

______, "Seepage Control in Earth Dams," Embankment-Dam Engineering, Casagrande Volume, John Wiley and Sons, New York, NY, pp. 21-45. 1973.

______, "Seepage, Drainage, and Flow Nets," Third Edition, John Wiley. CFGB (1997). Internal Erosion: Typology, Detection, Repair. Barrages and Reservoirs No.6, Comité Francais des Grands Barrages, Le Bourget-du-lac Cedex. 1989.

Chapuis, Robert P., "Similarity of Internal Stability Criteria for Granular Soils," Canadian Geotechnical Journal, 29, 711-713. 1992.

______, Transportation Research Record 1089 Charles, J.A., General Report, Special Problems Associated with Earthfill Dams, Nineteenth International Congress on Large Dams, Florence, GR Q73, Vol. II, 1986, pp.1083-1198. 1997.

______, "Predicting the saturated hydraulic conductivity of sand and gravel using effective diameter and void ratio," Canadian Geotechnical Journal, Vol. 41, pp. 787-795. 2004.

Compton, J.R. and W.E. Strohm, Jr., "Compaction of Cohesionless Materials," U.S. Army WES, Vicksburg, MS, Miscellaneous Paper S-68-15. August 1968.

D'Appolonia, D.J., R.V. Whitman, and E. D'Appolonia, "Sand Compaction with Vibratory Rollers," Journal of Geotechnical Engineering, ASCE, Vol. 95, No. SM1, pp. 263-284. January 1969.

Daniel, Dave E. and Robert M. Koerner, "Technical Guidance Document: Quality Assurance and Quality Control for Waste Containment Facilities," Chapter 2, EPA/600/R-93/182. September 1993.

Day, Steven R., Christopher R. Ryan , and Gary Fisk, Innovative Slurry Trench Methods for the Rehabilitation of Small Dams, 2001 ASDSO Annual Conference.

DM-7, "Soil Mechanics," NAVFAC DM 7.01, Naval Facilities Engineering Command Design Manual 7.01, United States Navy. September 1986.

Drnevich, Vincent P., Vibratory Hammer Method for Compaction of Granular Soils," ASTM Committee D18.03, Tampa, FL, ASTM Standard Test Method D7382, Standard Test Methods for Determination of Maximum Dry Unit Weight and Water Content Range for Effective Compaction of Granular Soils Using a Vibrating Hammer. January 12, 2004.

Drnevich, V.P. and A.B. Prochaska, "One-Point Vibrating Hammer Compaction Test for Granular Soils," Geotechnical Special Publications 130-142 & GRI-18; Proceedings of the Geo-Frontiers 2005 Congress, Austin, TX. January 24-26, 2005.

Dunnigan, Lorn P, "Design of Seepage Control Filters for Hydraulic Structures," 1988 Internal Summer Meeting of ASAE, Rapid City, SC, Paper #88-2074. June 1988.

Fannin, Jonathan, "On the Internal Stability of Granular Soils," Canadian Dam Association 2002 Annual Conference, Victoria, BC. October 2002.

______, "Karl Terzaghi: Path of Discovery to Filter Design," Journal of Geotechnical Engineering ASCE. March 2008.

Farrar, J.A. R.L. Torres, and Z. Erdogon, "Bureau of Reclamation Erosion Testing for Evaluation of Piping and Internal Erosion of Dams," Geotechnics of Soil Erosion, GSP 167, Proceedings of Geo-Denver. 2007.

Federal Emergency Management Agency (FEMA), Report of Workshop on Issues, Solutions and Research Needs Related to Seepage through Embankment dams, FEMA sponsored workshop. October 2000.

______, Technical Manual: Conduits through Embankment Dams, FEMA 484CD. September 2005.

______, Technical Manual: Plastic Pipe Used in Embankment Dams, FEMAP-675. November 2007.

______, Technical Manual: Embankment Dam Seepage – Best Practices for Monitoring, Measurement, and Evaluation, DRAFT. November 2009.

Federal Highway Administration, Integrated Materials and Construction Practices for Concrete Pavement: A State -of-the-Practice Manual, Chapter 3, Materials. FHWA Publication No. HIF-07-004, U.S. Department of Transportation. December 2006.

Fell, Robin and M. Foster, "The Internal Erosion and Piping Process, "Proceedings, Workshop on Issues, Solutions and Research Needs Related to Seepage through Embankment Dams," FEMA, Denver, CO. October 17-19, 2000.

Ferguson, Keith, Richard Hannan, James Talbot, and Francke Walberg, "Assessment and Repair of Active Foundation Piping and Erosion at Fern Ridge Dam, Oregon. Proceedings, Annual Meeting USSD, San Antonio, TX. 2006.

Fisk, Gary, J. Gagliardi, and S. Day, "Slurry Trench Construction of a Vertical Foundation Filter at Wasco Dam," Proceedings of the USSD Annual Conference, Denver, CO. 2001.

Foster, M., Robin Fell, and Matt Spannagle, A method for assessing the relative likelihood of failure of embankment dams by piping, Canadian Geotechnical Journal, 37:1025–1061. 2000a.

______, "The statistics of embankment dam failures and accidents," Canadian Geotechnical Journal, Vol. 37, pp. 1000-1024. 2000b.

Foster, M. A. and R. Fell, "Assessing Embankment Dams Filters which do not Satisfy Design Criteria", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 4, 398-407. 2001.

France, John W., "Report on Workshop on Issues, Solutions, and Research Needs Related to Seepage through Embankment Dams, 2001 ASDSO Annual Conference.

______, "Seepage and Piping – the Devil is in the Details," 2004 ASDSO Annual Conference, Phoenix, AZ.

______, "Seepage Collection and Control Systems: The Devil is in the Details," 2004 ASDSO Annual Conference, Phoenix, AZ.

Garner, S.J. and J.C. Sobkowicz, "Internal Instability in Gap-Graded Cores and Filters, Canadian Dam Association 2002 Annual Conference. October 2002.

Gerhart, Phil C., Steven R. Day, and Eric R. Dixon, A Bio-Polymer Drain Solution for Piute Dam, 2005 ASDSO Annual Conference.

Giroud, J.P., "Geosynthetics in Dams – Forty Years of Experience," 2005 ASDO Annual Conference, New Orleans, LA/Orlando, FL, unpublished presentation.

Goldsworthy, M.H., "Filter Tests—Direct or Indirect?" Geotechnique 40, No. 2, pp. 281-284. 1990.

Guy, Erich, Todd Calhoun, George Alford, Charles Barry, and Alford Rogers, "Relief Wells Rehabilitation at Alum Creek and Dillon Dams," 2006 ASDSO Annual Conference.

Hains, Colonel Peter C., Hearings before the Senate Committee on Interoceanic Canals on H.R. 3110, First Session, Fifty-Seventh Congress, in Three Parts, Testimony of Colonel Hains is on page 613. March 14, 1902.

Hamblin, Kenneth W., "The Earth's Dynamic Systems," Third Edition, Burgess Publishing Co., Minneapolis, MN. 1982.

Hammer, David P., "Construction of Vertical and Inclined Sand Drains in Embankment Dams, 2003 ASDSO Annual Conference, Minneapolis, MN. 2003.

Harza, L.G., "Uplift and Seepage Under Dams on Sand," Paper No. 1920, Transactions of ASCE. September 1934.

Hillis, S.F. and E.G. Truscott ,"Magat Dams: Design of Internal Filters and Drains," Canadian Geotechnical Journal: 20, 491-501. 1983.

Hirschfeld, Ronald, "Design, Construction, and Inspection of Embankment Dams to Reduce the Risk of Piping," Seepage, Piping and Remedial Measures, Interagency Committee on Dam Safety (ICODS) article, March 23-24, 1995.

Hjeldnes, E.I. and V.K. Lavania, Cracking, leakage and erosion of earth dam materials, Journal of the Geotechnical Engineering Division, ASCE. 106 (GT2), pp. 117-135. 1980.

Holtz, W.G., "The Relative Density Approach—Uses, Testing Requirements, Reliability, and Shortcomings," ASTM STP 523, Evaluation of Relative Density and its Role (sic) in Geotechnical Projects Involving Cohesionless Soils, ASTM, Philadelphia PA, pp. 5-16. July 1973.

Honjo, Y. and D. Veneziano, Improved filter criterion for cohesionless soils, Journal of Geotechnical Engineering, ASCE, 115 (1), pp. 75-94. 1989.

Hsu, S.J.C., "Aspects of Piping Resistance to Seepage in Clayey Soils," International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Vol. 10(1), 421-428. 1981.

Hurcomb, D., "Petrographic Examination of Exhumed Filter Sand - Horsetooth Dam Modifications - Colorado-Big Thompson Project," Earth Sciences and Research Laboratory Referral No. 8340-01-36. 2001.

ICOLD, Bulletin 95 -Embankment Dams Granular Filters and Drains, Review and Recommendations, ICOLD. 1994.

Indiana Department of Transportation, "Inspection and Sampling Procedures for Fine and Coarse Aggregate." September 2005.

Indraratna, B. and S. Radampola, "Analysis of Critical Hydraulic Gradient for Particle Movement in Filtration," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 4,347-350. 2002.

Johnson, A.I., "Discussion of Research on Vibratory Maximum Density Test for Cohesionless Soils," In: Compaction of Soils: ASTM Special Publication No. 377, Philadelphia: American Society for Testing and Materials, pp. 22-23. 1965.

Justin, Joel. D., "Discussion of Paper 1920, Transactions of ASCE titled "Uplift and Seepage under Dams on Sand" by L.F. Harza. September 1934.

Justin, Joel D., J. Hinds, and W.P. Creager, Engineering for Dams, Vol. 111," Earth, Rock-Fill, Steel and Timber Dams," John Wiley and Sons. 1945.

Kannik, Mia, A Boil on the Back Side of Salt Fork Lake Dam, What a Pain in the ... , 2005 ASDSO Annual Conference.

Karpoff, K.P., The use of laboratory tests to develop design criteria for protective filters, Proceedings, American Society of Testing Materials, Vol. 55, pp. 1183-1198. 1955.

Kassif, G., D. Zaslavsky, and J. Zeitlin, "Analysis of Filter Requirements for Compacted Clays," Proceedings, Sixth International Conference Soil Mechanics and Foundation Engineering, Montreal, Canada, Vol. 2, pp. 495-499. September 1965.

Kenney, T.C., Discussion of: Design of filters for clay cores of dams, Journal of the Geotechnical Engineering Division, ASCE, Vol. 109, No. 9, pp. 1191-1193. 1983.

Kenney, T.C., R. Chahal, E. Chiu, G.I. Ofoegbu, G.N. Omange, and C.A. Ume, "Controlling Constriction Sizes of Granular Filters," Canadian Geotechnical Journal: 22, 32-34. 1985.

Kenney, T.C. and D. Lau, "Internal Stability of Granular Filters," Canadian Geotechnical Journal: 22, pp. 2 15-225. 1985.

______, Closure to: Internal stability of granular filters, Canadian Geotechnical Journal, Vol. 23, pp. 420-423, 1986.

Khor, C.H. and H.K. Woo, Investigation of crushed rock filters for dam embankment, Journal of Geotechnical Engineering, ASCE, 115 (3), pp. 399-412. 1989.

Kleiner, David E., Use of Granular Filters and Drains in Embankment Dams, USCOLD Committee on Materials for Embankment Dams. 1994.

______, "A Review of Filter Criteria-Embankment Dams," Twenty-Second Congress on Large Dams, Barcelona, Spain. 2006.

Lade, P.V., C.D. Liggio, Jr., and J.A. Yamamuro, "Effects of Non-Plastic Fines on Minimum and Maximum Void Ratios of Sand," Geotechnical Testing Journal, Vol. 21, No. 4, pp. 336-347. 1998.

Lafleur, Jean, "Filter Testing of Broadly Graded Cohesionless Tills," Canadian Geotechnical Journal: 21, 634-643. 1984.

Lafleur, Jean, Jacek Mlynarek, and Andre' Rollin, "Filtration of Broadly Graded Cohesionless Soils," Journal of Geotechnical Engineering, Vol. 115, No. 12, pp. 1747-1768. December 1989.

Lee, Kenneth and Awtar Singh, "Relative Density and Relative Compaction," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM7, Technical Note, p. 1049-1052. July 1971.

Leonards, G.A., A.B. Huang, and J. Ramos, "Piping and Erosion Tests at Conner Run Dam," Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 1, 108-117. 1991.

Lo, K.Y. and K. Kaniaru, Hydraulic fracture in earth and rock-fill dams, Canadian Geotechnical Journal: 27, pp. 496-506. 1990.

Locke, M., B. Indraratna and G. Adikari, "Erosion and Filtration of Cohesive Soils," Filters and Drainage in Geotechnical and Environmental Engineering (ISBN: 90 5809 146 5) / GeoFilters 2000, 175-182. 2000.

Locke, M. and B. Indraratna, "Filtration of broadly graded soils: the PSD Method," Technical Note, Geotechnique, Vol. 52, No. 4, pp. 285-287. 2001.

Lone, M., A. Hussain, and G.L. Asawa, "Filter Design Criteria for Graded Cohesionless Bases," Journal GTE, ASCE, pp. 251-259. February 2005.

Lowe, John, III, “Fourth Nabor Carillo Lecture, Foundation Design – Tarbela Dam,” presented at the Mexican Society for Soil Mechanics X National Meeting, Merida, Yucatan, Mexico. November 18, 1978.

Luehring, Ron , Stuart Bezanson, and Roy Grant, Averting Incipient Piping Failure of a Turn-of-the-Century Wilderness Dam, 1999 ASDSO Annual Conference.

Mansur, Charles, George Postol, and J.R. Salley, “Performance of Relief Well Systems along Mississippi River Levees,” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 8, pp 727-738. August 2000.

McCook, Danny K., “Correlations Between a Simple Field Test and Relative Density Test Values,” Technical Note, American Society of Civil Engineering, Journal of Geotechnical Engineering, Vol. 122, No. 10, pp 860-862. October 1996.

______, “Chimney Filters – Design and Construction Considerations.” 1997 ASDSO Annual Conference, Pittsburg, PA.

______, “Things Your Mother Never Told You About Earth Dam Design.” 2002 ASDSO Annual Conference, Tampa, FL.

______, “The Filter Design Tightrope,” 2003 ASDSO Annual Conference, Minneapolis, MN.

______, “A Comprehensive Discussion of Piping and Internal Erosion.” 2004 ASDSO Annual Conference, Phoenix, AZ.

______, “Supplemental Tests to Evaluate Suitability of Materials Proposed for Use in Critical Filter Zones.” 2005 ASDSO Annual Conference, Orlando, FL.

McCook, Danny K. and Mark Pabst, Pipes in Drain Systems – Are They a Necessary Evil? 2006 ASDSO Annual Conference, Boston, MA.

McCook, Danny K. and J. Talbot, “NRCS Filter Design Criteria — A Step by Step Approach.” 1995 ASDSO Annual Conference, Atlanta, GA.

Middlebrooks, T.A. and William H. Jervis, “Relief Wells for Dams and Levees,” Transactions, ASCE, Paper No. 2327, Vol. 112, pp 1321-1399. June 1946.

Miller, Debora J., Alan C. Jewell, and Kevin Smith, Tongue River Dam. 2001 ASDSO Annual Conference.

Miller, Debora J., Dean B. Durkee, Michael A. Morrison, David B. Wilson, and Kevin Smith, Geosynthetics Applications and Performance Reviews – Select Case Histories. 2005 ASDSO Annual Conference.

Miller, Debora J. and Elaine C. Pacheco, “Emergency Seepage Mitigation for Cabresto Lake Dam, Questa, New Mexico. 2006 ASDSO Annual Conference.

Milligan, Victor, “Discussion of Kenney and Lau Article on Internal Stability of Granular Filters,” Canadian Geotechnical Journal, Vol. 23, pp 414-418. 1986.

______, "Some Uncertainties in Embankment Dam Engineering," The 38th Terzaghi Lecture. Proceedings of the 2002 Civil Engineering Conference and Exposition, Washington, DC. November 2002.

______, "Some Uncertainties in Embankment Dam Engineering," Journal of Geotechnical and Geonvironmental Engineering, ASCE, pp. 785-797. September 2003.

Mirata, Turker and Tanfer Gurler, "A Modified Way of Testing Filters for Clay Cores of Embankment Dams," Electronic Journal of Geotechnical Engineering. 2004.

Moffatt, R.A., "A Large Permeameter for Study of Internal Stability in Cohesionless Soils," Geotechnical Testing Journal ASTM 5, 7 p. April 2006.

Natural Resources Conservation Service, National Engineering Handbook, Chapter 5 (NEH-5), "Hydraulics." 1955.

______, Structural Design of Underground Conduits, National Engineering Handbook, Technical Release No. 5. 1958.

______, Technical Release 29 (TR-29), Hydraulics of two-way covered risers. 1965.

______, Design Note 8 (DN-08), Drop Inlet Entrance Losses. 1969.

______, USDA, Soil Mechanics Note SM-3, "Soil Mechanics Considerations for Embankment Drains." May 1971.

______, "Soil Mechanics Note SMN-5, Flow Net Construction and Use." 1973.

______, "Soil Mechanics Note SMN-7, The Mechanics of Seepage Analyses." October 1979.

______, "Permeability of Selected Clean Sands and Gravels," U.S. Department of Agriculture, National Resource Conservation Service, formerly Soil Conservation Service, Engineering Division, Soil Mechanics Note No. 9. March 1984.

______, Engineering Division, Soil Mechanics Note No. 1, 210-VI, "Guide for Determining the Gradation of Sand and Gravel Filters." Revised January 1986.

______, USDA. Soil Mechanics Note SMN-13, Dispersive Clays. February 1991.

______, "Gradation Design of Sand and Gravel Filters," U.S. Department of Agriculture National Engineering Handbook, Part 633, Chapter 26, Washington, DC. October 1994.

______, Alabama, Standard Drawings for Siphon Spillway Design. 1997.

______, USDA. National Engineering Handbook, Part 642, Chapter 26, "Construction Specification 24, Drainfill." 2001.

______, National Engineering Handbook, Part 636 Chapter 52, "Structural Design of Flexible Conduits." 2004.

Navin, Emily S., "Design and Installation of filters for Embankment Dams," Report of a Study Performed by the Virginia Polytechnic University Center for Practice and Research, Blacksburg, VA. November 2006.

Netherlands Committee on Flood Defence, Rijkswaterstaat, Road and Hydraulic Engineering Institute, P.O. Box 5044, 2600 GA Delft, The Netherlands, Technical Report on Sand Boils and Piping. March 1999.

Nettles, E.H. and N.B. Schomaker, "Laboratory Investigation of Soil Infiltration through Pipe Joints, Highway Research Board, 46th Annual Meeting, Washington DC, pp. 37-56. 1967.

Pabst, Mark, "Ochoco Dam Drain Rehabilitation." 1999 ASDSO Annual Conference, St. Louis, MO.

Pabst, Mark and J. France, "Getting Water in Pipes – Not as Easy as it Seems," 2007 ASDSO Annual Conference, Austin, TX.

Pabst, Mark, "Design and Construction of Access Features for Toe Drains," 2007b ASDSO Annual Conference, Austin, TX.

______, "Considerations for Filter Design," Proceedings of the USSD Annual Conference, 2007c, Denver, CO.

______, "Adding Filter Protection to Existing Conduits through Embankment Dams, 2009 ASDSO Annual Conference, Hollywood, FL.

Park, Youngjin, "Investigation of the Ability of Filters to Stop Erosion through Cracks in Dams," PhD thesis VPI, Blacksburg, VA. 2003.

Peck, Ralph B., private communication. 1973.

______, Where has all the judgment gone? Canadian Geotechnical Journal: 17 (4), pp. 584-590, 1980.

_______, "Interface between core and downstream filter," Proc., H. Bolton Seed Memorial Symp., Vol . 2, BiTech Publishers, Vancouver, 237-251. 1990.

Peck, R. B., W. E. Hansen, and T. H. Thornburn, "Foundation Engineering," Second Edition, John Wiley and Sons. 1974.

Perry, E.P., Piping in earth dams constructed of dispersive clay: literature review and design of laboratory tests, Technical Report S-75-15, Vicksburg, MS: Waterways Exp. Station. 1975.

______, "Laboratory Tests on Granular Filters for Embankment Dams," Technical Report GL 87-22, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS. August 1987.

Perry, Edward B. and Mosaid M. Al-Hussaini, "Filters and Drains for Earth Dams. 1995 ASDSO Annual Conference.

Pettibone, H.C. and J. Hardin, “Research on Vibratory Maximum Density Test for Cohesionless Soils,” In: Compaction of Soils: ASTM Special Publication No. 377, Philadelphia: American Society for Testing and Materials, pp. 3-19. 1965.

Pinto, P.S., E. Sêco, and T. Santana, “Filters for Clay Cores of Embankment Dams.” Twelfth International Conference on Soil Mechanics and Foundation Engineering, Vol. 12(3), 1689-1692. 1989.

Poulos, Steve, “Compaction Control and the Index Unit Weight,” Geotechnical Testing Journal, GTJODJ, Vol. 11, No. 2, pp. 100-108. June 1988.

Pritchett, E.C., “Embankment Seepage Control Design and Construction,” Seepage and Leakage from Dams and Impoundments, Volpe, R.L. and W.E. Kelly, eds., American Society of Civil Engineers, New York, NY, pp. 31-43. 1985.

Prochaska, Adam Buser, “An Alternative Method for Effective Compaction Control of Granular Soils, Thesis for Master Science Degree, Purdue University. August 2004.

Reddi, L.N., X. Ming, M.G. Hajra, and I.M. Lee,”Permeability Reduction of Soil Filters Due to Physical Clogging,” Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 3,236-246. 2000.

Ripley, C.F., Discussion published in the May 1982 ASCE Journal of Geotechnical Engineering. Re: article by Peter R. Vaughan and Hermusia F. Soares, “Design of Filters for Clay Cores of Dams,” Journal of Geotechnical Engineering. January 1982.

______, “Discussion Internal Stability of Granular Filters,” Canadian Geotechnical Journal, Vol. 23, pp. 255-258. 1986.

Ritter, Dale F., “Process Geomorphology,” W.M.C. Publishers, Dubuque, IA. 1978.

Saber, Robert T. and William J. Franz, “Evaluation of Broadly Graded Soils and Existing Filter Material for Compatibility and Internal Stability at G.B. Stevenson Dam. 2003 ASDSO Annual Conference.

Sakamoto, T. and Y. Yamaguchi, “Identification of Non-cohesive Filters to Prevent Migration of Impervious Materials for Embankment Dams due to Large Earthquakes (fault movement). Public Works Research Institute, Japan. 2002.

Schmertmann, John H., “The No-Filter Factor of Safety Against Piping through Sands,” Judgement and Innovation, Geotechnical Special Publication No. 111, ASCE, pp. 63-133. 2001.

______, Discussion of “A Method for Assessing the Relative Likelihood of Failure of Embankment Dams by Piping” Canadian Geotechnical Journal, Vol. 39: 495–496, original article by M. Foster, R. Fell, and M. Spannagle, Canadian Geotechnical Journal: 37: 1025–1061. 2002.

Selig, E.T. and R.S. Ladd, “Evaluation of Relative Density Measurements and Applications,” Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils, ASTM STP 523, pp. 487-504. 1973.

Shaqour, Fathi M., “Hydro-compaction of Sand Fills,” Electronic Journal of Geotechnical Engineering, Paper Number 0445. 2004.

Sherard, James L., “Embankment dam cracking,” In: Embankment Dam Engineering, Hirschfeld and Poulos eds., John Wiley & Sons, New York, NY, pp. 271-353. 1973.

______, “Sinkholes in Dams of Coarse, Broadly Graded Soils,” ICOLD, 13th Congress on Large Dams, Q.49, R2, New Delhi. 1979.

______, Discussion of: Design of filters for clay cores of dams. Journal of Geotechnical Engineering, ASCE, 109 (9), pp. 1195-1197. 1983.

______, Lecture on “Debatable Trends in Embankment Engineering,” presentation to the ASCE National Capital Section Geotechnical Committee, Seminar on Lessons Learned from Geotechnical Failures. February 3, 1984.

________, “Hydraulic Fracturing in Embankment Dams,” Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 10, pp. 905-927. October 1986.

Sherard, James L., et al., “Earth and Earth – Rock Dams”, John Wiley & Sons, Inc., New York, NY. 1963.

Sherard, J.L., R.S. Decker, and N.L. Ryker, Study of Piping Failures and Erosion Damage from Rain in Clay Dams in Oklahoma and Mississippi, a report prepared for the Soil Conservation Service. 1972.

______, “Hydraulic Fracturing in Low Dams of Dispersive Clay,” Proceedings Conference on the Performance of Earth and Earth-Supported Structures, ASCE, 905-927. June 1972a.

______, “Piping in Earth Dams of Dispersive Clay,” Proceedings Conference on the Performance of Earth and Earth-Supported Structures, ASCE, Vol. 1. 589-620. June 1972b.

Sherard, J.L., L.P. Dunnigan, and J.R. Talbot, “Basic Properties of Sand and Gravel Filters.” Journal of Geotechnical Engineering, ASCE, Vol. 110, No. 6, pp. 684-700. June 1984a.

______, “Filters for Silts and Clays.” ASCE, Journal of Geotechnical Engineering., ASCE, No. 110 (6). pp. 701-718. June 1984b.

Sherard, J.L. and Lorn P. Dunnigan, “Filters and Leakage Control in Embankment Dams,” Seepage and Leakage from Dams and Impoundments, ASCE Symposium, Denver, CO,
pp. 1-29. May 1985.

______, “Discussion on Internal Stability of Granular Filters,” Canadian Geotechnical Journal, Vol. 23, pp. 418-420. 1986.

______, “Critical Filters for Impervious Soils,” ASCE, Journal of the Geotechnical Engineering Division, No. 115(7), pp. 927-947. July 1989.

Sherman, W.C., Filter Concepts and Design Criteria, unpublished paper, NTIS AD 771076, Waterways Experiment Station, Vicksburg, MS. 1953.

Silveria, A., "An Analysis of the Problem of Washing through in Protective Filters," Proceedings, 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, CA, Vol. 2. 1965.

Singh, V.P., et al., "Role of Sand Boil Formation in Levee Failure," paper presented at XXIX International Association of Hydraulic Engineering and Research, Theme C. Beijing, China. September 2001.

Skempton, A. and J.M Brogan, "Experiments on Piping in Sandy Gravels, Geotechnique, Vol. 44, No. 3, pp 449-460. 1994.

Slichter, Charles S., "Field Measurements of the Rate of Movement of Underground Waters," United States Geological Survey Water Supply and Irrigation Paper No. 140. GPO, Washington, DC. 1905.

Smith Stuart A. and Denise M. Hosler, Current Research in Dam Drain Clogging and Its Prevention. 2006 ASDSO Annual Conference.

Spangler, M.G. and R.L. Handy, Soil Engineering, 4th Edition, Harper and Row. 1982.

Stearns, Frederic P., "Discussion on the Bohio Dam, including Filter Experiments," Transactions of the ASCE, Vol. 48, pp. 257-279. 1902.

Sutherland, Kenneth and M. Grabinsky, "Evaluating Segregation in Ganular Filters for Earth Dams," 56th Canadian Geotechnical Conference, Vol. 1, p. 151. 2003.

Talbot, James R. and David C. Ralston, "Earth Dam Seepage Control, SCS Experience." Seepage and Leakage from Dams and Impoundments, ASCE Symposium, p. 4, Denver, CO. May 1985.

Talbot, James and Mark Pabst, "Filters for Earth Dams Gradation Design and Construction Guidance Used by Federal Agencies," ASDSO Journal of Dam Safety, pp. 13-24. Winter, 2006.

Tavenas, F.A., "Difficulties in the Use of Relative Density as a Soil Parameter" ASTM STP 523, Evaluation of Relative Density and its Role (sic) in Geotechnical Projects Involving Cohesionless Soils, ASTM, Philadelphia, PA, pp. 478-483. July 1973.

Tavenas, F.A., R.S. Ladd, and P. La Rochelle, "Accuracy of Relative Density Measurements; Results of a Comparative Test Program," ASTM STP 523, Evaluation of Relative Density and its Role (sic) in Geotechnical Projects Involving Cohesionless Soils: ASTM, Philadelphia, PA, pp. 18-50. July 1973.

Taylor, Donald W., Fundamentals of Soil Mechanics, John Wiley & Sons, London. 1948.

Terzaghi, K. , Journal of the Boston Society of Civil Engineers, Vol. XII, , Number 10, "Modern Conceptions Concerning Foundation Engineering." December 1925.

_______, "Discussion of Paper 1920, Transactions of ASCE titled "Uplift and Seepage under Dams on Sand" by L.F. Harza. September 1934.

Terzaghi, K. and R.B. Peck, Soil Mechanics in Engineering Practice. Second edition, John Wiley. 1967.

Terzaghi, K., R.B. Peck, and G. Mesri, "Soil Mechanics in Engineering Practice," 3rd edition, Wiley, New York, NY. 1996.

Thanikachalam, V. and R. Sakthivadivel, "Rational Design Criteria for Protective Filters," Canadian Geotechnical Journal, Vol. 11. 1974.

Tiedemann, D.A., "Variability of Laboratory Relative Density Test Results." In: E.T. Selig and R.S. Ladd (Eds.), Evaluation of Relative Density and its Role in Geotechnical Projects Involving Cohesionless Soils: ASTM Special Technical Publication No. 523, pp. 61-73. Philadelphia: American Society for Testing and Materials. 1973.

Tomlinson, S.S. and Y.P. Vaid, "Seepage forces and confining pressure effects on piping erosion, Canadian Geotechnical Journal, Vol. 37, pp. 1-13. 2000.

Townsend, F.C., J.M. Shiau, and T.J. Pietrus, "Piping Susceptibility and Filter Criteria for Sand," Engineering Aspects of Soil Erosion, Dispersive Clays and Loess, Geotechnical Special Publication 10, ASCE, 46-66. 1987.

Truscott, E.G., Discussion of: "Design of filters for clay cores of dams," Journal of Geotechnical Engineering, ASCE, 109 (9), pp. 1195-1197. September 1982

Turnbull, W.J. and C.I. Mansur, "Design of Control Measures for Dams and Levees," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, ASCE, 345 East 47th Street, New York, NY, 10017. October 1959.

______, "Construction and Maintenance of Underseepage Control Measures," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, ASCE, 345 East 47th Street, New York, NY, 10017. December 1959.

U.S. Army Corps of Engineers, Waterways Experiment Station, Investigation of Filter Requirements for Underdrains, Technical Memorandum No. 1 83 - 1, Vicksburg, MS. November 1941 (revised December 1941).

______, "Subsurface Drainage Investigation, 1945-1946 Comprehensive Report," U.S. Army Engineers District, CE, Boston, MA. 1945-1946.

______, Waterways Experiment Station, Laboratory Investigation of Filters for Enid and Grenada Dams, Technical Memorandum No. 3 k-245, Vicksburg, MS. January 1948.

______, Waterways Experiment Station, Filter Experiments and Design Criteria, Technical Memorandum No. 3-360, Vicksburg, MS. 1953.

______, Waterways Experiment Station, Investigation of Underseepage and Its Control, Lower Mississippi River Levees, Technical Memorandum No. 3-424, Vicksburg, MS. October 1956.

______, "Underseepage and Its control," ASCE Symposium, Journal of the Soil Mechanics and Foundation Division, p. 1513. August 1959.

______, Hydraulic Design of Reservoir Outlet Works, EM 1110-2-1602. 1980.

______, Laboratory Tests on Granular Filters for Embankment Dams, Technical Report GL-87-22, Waterways Experiment Station, Vicksburg, MS. 1987.

______, Engineer Manual 1110-2-1914, "Design, Construction, and Maintenance of Relief Wells," Washington, DC. May 1992.

______, Engineer Manual EM-1110-2-1913, "Design and Construction of Levees," Washington, DC. April 30, 2000.

______, Engineer Manual EM 1110-1-1804, Geologic Investigations, Appendix F-12, "Sampling from Stockpiles and Bins, Transportation Units, or Conveyor Belts." Washington, DC. January 2001a.

______, Technologies and Techniques for Drainage Pipe Maintenance and Repair: A Practical Guide to Technology Selection. 2001b.

______, Engineer Manual 1110-2-2300, General Design and construction Considerations for Earth and Rock-Fill Dams, Washington, DC. 2004.

______, Engineer Manual EM 1110-2-1901, Seepage Control, Appendix D, Filter Design," Washington, DC. 2004.

______, Engineer Manual EM 1110-2-1901, "Embankment Seepage Control," Appendix D, "Filter Design," Washington, DC. May 2005.

USCOLD, Lessons from dam incidents, USA. Committee on Failures and Accidents to Large Dams of the United States Committee on Large Dams (USCOLD), American Society of Civil Engineers, New York, USCOLD. 1988. Lessons from dam incidents, USA. II. Committee on Dam Safety of the United States Committee on Large Dams. 1975.

Vaughan, P.R. and R.C. Bridle, "An Update on Perfect Filters-Corrected," Long-Term Benefits and Performance of Dams, Thomas Telford, London, England. 2005.

Vaughan, P.R. and H.F. Soares, "Design of Filters for Clay Cores of Dams," Journal of the Geotechnical Division, ASCE, 108 (GT1), pp. 17-31. January 1982.

Virginia Polytechnic Institute. Andrew Bolton, Thomas Brandon, J. Michael Duncan, and James K. Mitchell, "Soil Slump Index Test," USBR Contract No. 05PG810038. 2008.

Von Thun, J.L., Understanding seepage and piping failures – the No. 1 dam safety problem in the west. In: Proceedings of the Association of Dam Safety Officials (ASDSO) Western Regional Conference, Lake Tahoe, NV, pp. 1-24. 1996.

Weijers, J.B.A. and J.B. Sellmeijer. A new model to deal with the piping mechanisms. Filters in Geotechnical and Hydraulic Engineering, Brauns, Heibaum & Schuler (eds.), Balkema, Rotterdam, pp. 349-355. 1993.

Wilson, L.C. and A.L. Melvill, Discussion of: "Design of filters for clay cores of dams," Journal of Geotechnical Engineering, ASCE, 109 (9), pp. 1198-1200. 1983.

Wolski, W., Model Tests on the Seepage Erosion in the Silty Clay Core of an Earth Dam. Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal, Vol. II, Division 3-6, pp. 583-587. 8-15 September 1965.

Youd, T.L., "Factors Controlling Maximum and Minimum Densities of Sands," Evaluation of Relative Density and its Role in Geotechnical Projects Involving Cohesionless Soils, ASTM STP 523, pp. 98-112. June 1972.

Zaslavsky, D. and G. Kassif, "Theoretical Formulation of Piping Mechanism in Cohesive Soils," Geotechnique, 15:3: 305. 1965.

Glossary

The terms defined in this glossary use industry-accepted definitions whenever possible. The source of the definition is indicated in parentheses. Some definitions may have been slightly modified to fit the context of this document.

Absorption – The increase in the weight of aggregate due to water in the pores of the material, but not including water adhering to the outside surface of the particles, expressed as a percentage of the dry weight. The aggregate is considered "dry" when it has been maintained at a temperature of 110 plus or minus 5 deg Celsius for sufficient time to remove all uncombined water.

Abutment – That part of the valley wall against which the dam is constructed. Left and right abutments are defined on the basis of looking in the downstream direction.

Anisotropy – Variability of a soil in that the horizontal permeability is not the same as the vertical permeability. Typically natural deposits and manmade fill will have greater horizontal than vertical permeability since they are placed in a horizontal fashion.

Angle of friction (ASTM D653, 2002) – Angle whose tangent is the ratio between the maximum value of shear stress that resists slippage between two solid bodies at rest with respect to each other, and the normal stress across the contact surface.

Apparent specific gravity – The ratio of the weight in air of a unit volume of the impermeable portion of aggregate at a stated temperature to the weight in air of an equal volume of gas-free distilled water at a stated temperature.

Arching – The soil property in which stresses distribute between a stiffer element, such as rock or concrete structure, and another stress plane or stiffer element, in such a way that the vertical stresses are less than the overburden pressure.

Backward erosion piping – Erosion of soil that begins from a concentrated seepage location usually in the downstream area of a dam. As the erosion continues more and more material is removed resulting in a pipe shaped void. This erosion continues upstream towards the highest gradient, or backward from the initiation point.

Base soil – The soil material that is being protected by a filter. Base soils are up gradient of the filter.

Bedrock – A general term that includes any of the generally indurated or crystalline materials that make up the Earth's crust. Individual stratigraphic units or units significant to engineering geology within bedrock may include poorly or nonindurated materials such as beds, lenses, or intercalations. These may be weak rock units or interbeds consisting of clay, silt, and sand.

Best practice – A generally recognized procedure preferred by the profession. A best practice is not necessarily a common practice, example: the inclusion of seepage collars around conduits.

Binding agents – Material, either mineral or chemical, that coat filter material resulting in that material being cemented or bound together.

Blanket – A horizontal layer or zone in an embankment dam between the downstream shell and foundation. It typically provides drainage from the chimney filter to the toe drain. Also see Drainage blanket.

Blanket drain – The second stage of a filter/drain blanket system consisting of primarily gravel-size material.

Blanket filter – The first stage of a filter/drain blanket system consisting of primarily sand-size material.

Blind well – A special type of relief well that does not include a casing or riser pipe.

Broadly graded – A characteristic of a soil gradation where a variety of soil grain sizes are present.

Bulk specific gravity (SSD) – The ratio of the weight in air of a unit volume of aggregate, including the weight of water within the voids filled to the extent achieved by submerging in water for approximately 24 hr (but not including the voids between the particles) at a stated temperature, compared to the weight in air of an equal volume of gas-free distilled water at a stated temperature.

Category 1 soil – Base soil that has more than 85% fines after regrading.

Category 2 soil – Base soil that has between 40 and 85% fines after regrading.

Category 3 soil – Base soil that has between 15 and 40% fines after regrading.

Category 4 soil – Base soil that has less than 15% fines after regrading.

Cementing agents – Chemicals (usually in solution form) that coat filter aggregate. These agents are not detected using grain size analysis and will not classify as fines using the USCS.

Chimney – A zone in an embankment dam that extends from the foundation to near the top of the dam. Chimneys can be vertical or inclined.

Chimney drain – The second stage of a filter/drain chimney system consisting of primarily gravel-size material.

Chimney filter – The first stage of a filter/drain chimney system consisting primarily of sand-size material.

Choke filter – See Inverted filter.

Clean – A soil gradation that contains less than 5% fines by weight.

Coefficient of curvature (also coefficient of gradation) – Determined from a grain-size analysis, calculated from the relationship: $C_z = D_{30}^2/(D_{60} * D_{10})$ where D_{60}, D_{30}, and D_{10} are the particle diameters corresponding to 60, 30, and 10% finer on the cumulative gradation curve, respectively.

Coefficient of gradation – See Coefficient of curvature.

Coefficient of internal friction (ASTM D653 2002) – The tangent of the angle of internal friction.

Coefficient of uniformity – Determined from a grain-size analysis, equal to the ratios D_{60} / D_{10}, where D_{60} and D_{10} are the particle diameters corresponding to 60 and 10% finer on the cumulative gradation curve, respectively.

Compaction (FEMA 2004) – Mechanical action that increases density by reducing the voids in a material.

- *End Result* – A compaction process that includes requirements for maximum lift thickness and other criteria to ensure that the compacted soil has the intended properties.
- *Method* – A compaction process that only specifies the equipment and its operation in compacting the soil.

Compactor – Machinery or device used to increase the density of soil. Also see Roller.

Conduit – Typically a pipe, box, or horseshoe structure that is constructed by means of "cut and cover." A conduit can convey water or house other conduits, pipes, cables, wires, etc.

Controlled low strength material (CLSM) – A self-compacting cementious material used primarily as backfill in lieu of compacted fill. CLSMs are defined as materials that result in a compressive strength of 1,200 lb/in.^{2} or less. Also known as flowable fill, unshrinkable fill, controlled density fill, and flowable mortar.

Core – In a zoned embankment, the core is the portion of the embankment having the lowest permeability and is intended to limit the quantity of seepage through the embankment to an acceptable amount.

Coverage – The amount of surface area that is compacted in one trip. For steel drum rollers, the coverage is 100%. For rubber-tire rollers, the coverage is 50% due to the space between the tires. Therefore, two passes/trips are required to obtain 100% coverage.

Crack – A long, narrow opening or a separation in previously intact material. Also see longitudinal and transverse crack.

Critical filter – A filter material used as a protective zone in an embankment dam, typically protecting the core. Filters also used to protect foundation soils, such as in toe drains are also critical filters. Filter material use as a transition zone, choke filter, or riprap bedding are examples of non-critical filters.

Critical gradient – The gradient at which seepage will cause soil particles to begin to move. In cases where seepage exits the ground surface vertically the critical gradient is calculated as unity.

Cutoff trench – An excavation in the foundation of an embankment dam below the original streambed elevation that is intended to reduce underseepage.

Cutoff wall – A vertical barrier under a dam usually constructed in a deep vertically sided trench. The backfill in the trench can be a variety of materials including concrete, soil-bentonite, and soil-cement-bentonite. A wall of impervious material (e.g., concrete, timber, steel sheet piling) located in the foundation beneath the dam, which forms a water barrier to reduce underseepage.

Dam (ICODS 2003) – An artificial barrier that has the ability to impound water, wastewater, or any liquid-borne material for the purpose of storage or control of water.

- *Earthfill (FEMA 2004)* – An embankment dam in which more than 50% of the total volume is formed of compacted earth layers comprised of material generally smaller than 3 in.
- *Embankment (FEMA 2004)* – Any dam constructed of excavated natural materials, such as both earthfill and rockfill dams, or of industrial waste materials, such as a tailings dams.

- *Rockfill (FEMA 2004)* – An embankment dam in which more than 50% of the total volume is comprised of compacted or dumped cobbles, boulders, rock fragments, or quarried rock generally larger than 3 in.
- *Tailings (FEMA, 2004)* – An industrial waste dam in which the waste materials come from mining operations or mineral processing.

Dam height – The vertical difference between the lowest point in the original streambed at the dam axis (or the crest centerline) and the crest of the dam.

Dead pool – Water that cannot be released from the reservoir through spillways, outlet works, powerplants, or all other designed appurtenances of the dam because the water lies below the invert elevation of the lowest water release feature at the dam.

Defect – An anomaly in an earthfill dam such as a crack, poorly placed lift, or separation between the fill and concrete structure.

Deformation (ACI 2000) – A change in dimension or shape due to stress.

Discharge face – The downstream face of the base soil through which seepage flow passes.

Discharge point – The end of a toe drain system where flow is discharged into some other water course or drainage way.

Dispersive soil – Soil that has higher than typical erosion potential due to its uncommon characteristic of dispersing into seepage flow.

Drain – Typically a second stage of a filter/drain system consisting of gravel. A feature designed to collect water and convey it to a discharge location. Typically, a drain is intended to relieve excess water pressures.

Drainage blanket – An embankment zone that provides drainage from the base of the chimney to the toe drain or foundation seepage.

Drainpipe – A system of pipe within an embankment dam used to collect seepage from the foundation and embankment and convey it to a free outlet.

Embankment dam (ICODS 2003) – Any dam constructed of excavated natural materials, such as both earthfill and rockfill dams, or of industrial waste materials, such as a tailings dams.

End result compaction – See Compaction, end result.

Erosion – Removal of soil grains by either surface water flow or seepage through the ground.

Failure – A circumstance in which uncontrolled releases of reservoir water from a dam occur that have an adverse impact on downstream persons or property.

Failure mode (FEMA 2004) – A physically plausible process for an embankment dam failure, resulting from an existing inadequacy or defect related to a natural foundation condition, the dam or appurtenant structure's design, the construction, the materials incorporated, the operation and maintenance, or aging process, which can lead to an uncontrolled release of the reservoir.

Filter – A zone of material designed and installed to provide drainage, yet prevent the movement of soil particles due to flowing water. A material or constructed zone of earthfill that is designed to permit the passage of flowing water through it, but prevents the passage of significant amounts of suspended solids through it by the flowing water.

- *Chimney* – A chimney filter is a vertical or near vertical element in an embankment dam that is placed immediately downstream of the dam's core. In the case of a homogenous embankment dam, the chimney filter is typically placed in the central portion of the dam.
- *Collar* – A limited placement of filter material that completely surrounds a conduit for a specified length within the embankment dam. The filter collar is located near the conduit's downstream end. The filter collar is usually included in embankment dam rehabilitation only when a filter diaphragm cannot be constructed. A filter collar is

different from a filter diaphragm in that a filter diaphragm is usually located within the interior of the embankment dam.

- *Diaphragm* – A filter diaphragm is a zone of filter material constructed as a diaphragm surrounding a conduit through an embankment. The filter diaphragm protects the embankment near the conduit from internal erosion by intercepting potential cracks in the earthfill near and surrounding the conduit. A filter diaphragm is intermediate in size between a chimney filter and a filter collar. The filter diaphragm is placed on all sides of the conduit and extends a specified distance into the embankment.

Filter cake – A thin layer of soil particles that accumulate at the face of a filter when water flowing through a crack carries eroding particles to the face. The filter cake forms when eroded particles embed themselves into the surface voids of the filter. The filter cake is effective in reducing further water flow to that which would occur through a layer of soil with the permeability of the eroded soil particles.

Filter collar – See Filter, collar.

Filter diaphragm – See Filter, diaphragm.

Fines – The soil grain sizes that are smaller than the No. 200 sieve (0.075 mm) as used in the USCS.

First filling – Usually refers to the initial filling of a reservoir or conduit. After major repairs, the re-filling of the reservoir may also be referred to a first filling.

First stage – The initial stage of a filter/drain system usually consisting of sand. The first stage protects foundation soils or impervious core.

Flexible pipe – A pipe that derives its load carrying capacity by deflects at least 2% into the surrounding medium upon application of load.

Flood (FEMA 2004) – A temporary rise in water surface elevation resulting in inundation of areas not normally covered by water. Hypothetical floods may be expressed in terms of average probability of exceedance per year, such as a 1-percent-chance flood, or expressed as a fraction of the probably maximum flood or other reference flood.

Forensics – The branch of science that employs scientific technology to assist in the determination of facts.

Foundation (FEMA 2004) – The portion of a valley floor that underlies and supports an embankment dam. *Soil or rock materials present at the damsite upon which a dam is built. Foundation materials that are consolidated into rock or rock-like material may be referred to as bedrock, while unconsolidated materials may be referred to as surficial materials.*

Freeboard – The difference in elevation between the maximum reservoir water surface and the dam crest.

Gap-graded – A soil property in which a particular soil grain size is missing from the central portion of the gradation curve, such as when no fine sand grain sizes are present in a sand and gravel soil, there is a "gap" in the fine sand size. Also known as skip-graded.

Geophysical techniques – Methods used to study the physical characteristics and properties of embankment dams. Geophysical techniques are based on the detection of contrasts in different physical properties of materials.

Geotextiles (FEMA 2004) – Any fabric or textile (natural or synthetic) when used as an engineering material in conjunction with soil, foundations, or rock. Geotextiles have the following uses: drainage, filtration, separation of materials, reinforcement, moisture barriers, and erosion protection.

Gradation (ASTM C 822 2002) – The distribution of particles of granular material among standard sizes usually expressed in terms of cumulative percentages larger or smaller than each of a series of sieve openings.

Gradation band – The range of particle sizes for which a filter gradation is specified. The gradation band must fit within the limits determined by the filter design procedure. Also see Limits.

Gradient – The change in head over a given distance. A descriptor of the potential for seepage water to move (erode) a soil particle.

Grain size distribution – A visual representation of the percentage of specified soil particle sizes relative to one another.

Gravel – Materials that will pass a 3-inch (76.2-mm) and be retained on a No. 4 (4.75-µm) U.S. standard sieve.

Groin – The line of contact between the face of the dam (upstream or downstream) and the abutment.

Ground-penetrating radar – A geophysical method that uses high-frequency radio waves to locate voids at shallow depths, less than about 15 to 20 ft (the effective depth is very limited in clayey soils).

Grout (FEMA 2004) – A fluidized material that is injected into soil, rock, concrete, or other construction material to seal openings and to lower the permeability and/or provide additional structural strength. There are four major types of grouting materials: chemical, cement, clay, and bitumen.

Grout mix (ASTM D 653 2002) – The proportions or amounts of the various materials used in the grout, expressed by weight or volume (the words "by volume" or "by weight" should be used to specify the mix).

Grout pipe – The pipe used to transport grout to a certain location. The grout may be transported through this pipe by either gravity flow or pressure injection.

Hazard (FEMA 2004) – A situation that creates the potential for adverse consequences such as loss of life or property damage.

Hazard potential classification – A system that categorizes embankment dams according to the degree of adverse incremental consequences of a failure or misoperation of a dam. The hazard potential classification does not reflect in any way on the current condition of the embankment dam (i.e., safety, structural integrity, flood routing capacity).

Head – The vertical difference, typically expressed in ft, between two water surface elevations.

Height (above ground) – The maximum height from natural ground surface to the top of an embankment dam.

Heterogeneous – Consisting of dissimilar constituents. For soils, consisting of several soil types.

High density polyethylene (HDPE) – A polymer prepared by the polymerization of ethylene as the sole monomer.

Homogeneous – Consisting of similar constituents. For soil, consisting of a single soil type.

Hydraulic conductivity – The ease at which water can flow though a soil. The *coefficient* of hydraulic conductivity is a property of a soil in which the waterflow through the soil is a function of the gradient and cross sectional area of the flow path.

Hydraulic fracture – A separation in a soil or rock mass that occurs if the applied water pressure exceeds the lateral effective stress on the soil element. Hydraulic fracture may occur if differential foundation movement is allowed. Soils compacted dry of optimum water content are more susceptible to hydraulic fracture.

Hydraulic gradient – The slope of the hydraulic grade line. The hydraulic gradient is the slope of the water surface in an open channel.

Hydraulic height – The vertical difference between the lowest point in the original streambed at the dam axis (or the centerline crest of the dam) and the maximum controllable water surface (which often is the crest of an uncontrolled overflow spillway).

Hydraulic structure – Any structure that retains or carries water (dams, levees, canals, spillways, retaining walls, etc.).

Hydrophilic – Having a strong affinity for water.

Hydrophobic – Having a strong aversion to water.

Hydrostatic head (ASTM D 653 2002) – The fluid pressure of water produced by the height of the water above a given point.

Hydrostatic pressure – The pressure exerted by water at rest.

Ice lens – A mass of ice formed during the construction of an embankment dam when a moist soil is exposed to freezing temperatures. In certain types of soils (silts and silty clay soils), the size of the ice mass will increase as it draws unfrozen capillary water from the adjacent soil. A void in the soil may remain after the ice lens melts.

Impervious – Not permeable; not allowing liquid to pass through.

Incident (ICOLD 1974) – Either a failure or accident that requires a major repair.

Inclined filter – A sloping embankment zone located near the control portion of the cross section. Also see Chimney.

Infiltration – The flow of water through a soil surface or the flow of water into a conduit through a joint or defect.

Inspection – The review and assessment of the operation, maintenance, and condition of a structure.

Inspector – The designated onsite representative responsible for inspection and acceptance, approval, or rejection of work performed as set forth in the contract specifications. The authorized person charged with the task of performing a physical examination and preparing documentation for inspection of the embankment dam and appurtenant structures.

Instrumentation (FEMA 2004) – An arrangement of devices installed into or near embankment dams that provide for measurements that can be used to evaluate the structural behavior and performance parameters of the structure.

Intergranular flow path – Flow of water through the voids or pore spaces of a soil.

Internal erosion – A general term used to describe all of the various erosional processes in which water moves internally through or adjacent to the soil zones of embankment dams and foundation, except for the specific process referred to as “backward erosion piping.” The term “internal erosion” is used in this document in place of a variety of terms that have been used to describe various erosional processes, such as scour, suffosion, concentrated leak piping, and others. A term used to describe the process of erosion of dam or foundation soils by flowing water, which includes erosion by such mechanisms as scour, internal instability of soils, heave, or “piping.”

Internal instability – A property of soil in which particles can move within the mass itself.

Inundation map (FEMA 2004) – A map showing areas that would be affected by flooding from releases from a dam’s reservoir. The flooding may be from either controlled or uncontrolled releases or as a result of a dam failure. A series of maps for a dam could show the incremental areas flooded by larger flood releases.

Inverted filter – A filter placed in reverse order in an effort to stop material erosion from a concentrated seepage area. The second stage (gravel) is placed first to attenuate the flow of water. Next, the first stage (sand) is placed to stop the material erosion.

Isotropy – Uniformity of a soil in that the horizontal permeability is the same as the vertical permeability.

Joint – The location at which two zones in an embankment dam come together.

Leakage (FEMA 2004) – Uncontrolled loss of water by flow through a hole or crack.

Lift – A soil layer of a given thickness placed during embankment construction.

Limits – The control points as determined by the design procedures in which a filter gradation must fit so filter criteria are met.

Liquefaction – A sudden loss of strength in saturated soils caused by an increase in pore pressure, which results from loose soils being subjected to earthquake shaking. This loss of strength in embankment or foundation soils could result in a slope failure of the dam.

Loess – Silt which transported by the wind over many miles, sometimes hundreds of miles and deposited in deposits in thickness of several inches to several hundred feet. Many loess deposits are non-plastic and have little erosion resistance.

Longitudinal crack – A crack in an embankment dam parallel to the axis (centerline) of the dam.

Maintenance – All routine and extraordinary work necessary to keep a facility in good repair and reliable working order to fulfill the intended designed project purposes. This includes maintaining structures and equipment in the intended operating condition and performing necessary equipment and minor structure repairs.

Maximum water surface – The highest acceptable water surface elevation considering all factors affecting a dam.

Method compaction – See Compaction, method.

Monitoring – The process of measuring, observing, or keeping track of something for a specific period of time or at specified intervals.

Moisture content – See Water content.

Multilayer filter – A filter/drain system consisting of more than one stage (i.e., a two-stage filter).

Nonpressurized flow – Open channel discharge at atmospheric pressure for part or all of the conduit length. This type of flow is also referred to as "free flow."

Normal water surface (FEMA 2004) – For a reservoir with a fixed overflow sill, this is the lowest crest level of that sill. For a reservoir whose outflow is controlled wholly or partly by moveable gates, siphons, or other means, it is the maximum level to which water may rise under normal operating conditions, exclusive of any provision for flood surcharge.

Nuclear gauge – An instrument used to measure the density and water content of both natural and compacted soil, rock, and concrete masses. The gauge obtains density and water contents from measurements of gamma rays and neutrons that are emitted from the meter. Gamma rays are emitted from a probe inserted into the mass being measured. Measurement of the gamma rays transmitted through the mass, when calibrated properly, reflects the density of the mass. Neutrons are emitted from the base of the gauge. Measuring the return of reflected neutrons when the gauge is calibrated properly can be related to the water content of the mass.

Offset (ACI 2000) – An abrupt change in alignment or dimension, either horizontally or vertically.

Open cut – An excavation through rock or soil made through topographic features.

Optimum moisture content (optimum water content) (ASTM D 653 2002) – The water content at which a soil can be compacted to a maximum dry unit weight by a given compactive effort.

Outlet works (FEMA 2004) – An embankment dam appurtenance that provides release of water (generally controlled) from a reservoir. *A pipe or conduit provided at a dam through which normal releases from the reservoir can be made.*

Overburden – The soil that overlies bedrock.

Passes – One trip for a single-drum roller. When a roller has two drums, one trip is equal to two passes.

Perforated pipe – A pipe intended to collect seepage through holes or slots on its exterior.

Permeability – The ease at which water or other fluid, including gasses, can flow through a material.

Pervious – Permeable, having openings that allow water to pass through.

Pervious zone (FEMA 2004) – A part of the cross section of an embankment dam comprising material of high permeability.

Phreatic line (ASCE 2000) – Water surface boundary. Below this line, soils are assumed to be saturated. Above this line, soils contain both gas and water within the pore spaces.

Phreatic surface (ASCE 2000) – The planar surface between the zone of saturation and the zone of aeration. Also known as free-water surface, free-water elevation, groundwater surface, and groundwater table. The top of the zone of saturation in an embankment. Seepage through the embankment causes the saturation, and the location of the phreatic surface typically varies in response to changing reservoir and tailwater conditions.

Piezometer (ASCE 2000) – An instrument for measuring fluid pressure (air or water) within soil, rock, or concrete. *A device for measuring the pore water pressure at a specific location in earthfill or foundation materials.*

Pipe – A hollow cylinder of concrete, plastic, or metal used for the conveyance of water.

- *Cast iron* – A type of iron-based metallic alloy pipe made by casting in a mold.
- *Corrugated metal* – A galvanized light gauge metal pipe that is ribbed to improve its strength.
- *Ductile iron* – A type of iron-based metallic alloy pipe that is wrought into shape.
- *Plastic (ASTM F412 2001)* – A hollow cylinder of plastic material in which the wall thicknesses are usually small when compared to the diameter and in which the inside and outside walls are essentially concentric.
- *Precast concrete* – Concrete pipe that is manufactured at a plant.

- *Steel* – A type of iron-based metallic alloy pipe having less carbon content than cast iron but more than ductile iron.

Piping – The removal of embankment or foundation material by flowing water through a cross section of limited size (initially) because of the ability of the embankment or foundation to provide a “roof” that does not significantly collapse into the developing “pipe.” Progresses upstream from a downstream exit location and can lead to dam failure if the developing “pipe” reaches the reservoir or if the enlarging pipe collapses and results in crest loss that leads to overtopping. Similar to subsurface erosion or internal erosion by seepage flow, except only true piping involves the capability to provide a “roof” that reduces the amount of embankment or foundation material that needs to be transported by the seepage flow to extend the flow path from the downstream exit to the reservoir. Also see Backward erosion piping.

Plastic pipe (ASTM F412 2001) – A hollow cylinder of plastic material in which the wall thicknesses are usually small when compared to the diameter and in which the inside and outside walls are essentially concentric.

Plasticity – A soil property indicating moldability or ability to remold.

Plasticity index – A measure of soil plasticity as determined using ASTM D4318. The mathematical difference between the liquid limit and plastic limit.

Poisson’s ratio – The soil property that is the ratio between strain changes in two orthogonal directions.

Pore pressure (ASCE 2000) – The interstitial pressure of a fluid (air or water) within a mass of soil, rock, or concrete.

Preferential flow path – A crack in a soil mass or a separation between soil and a structure or rock contact.

Pull-a-part – A geologic condition of foundation rock where geologic processes have result in tensile zones at the rock surface. These tensile zones result in large joint and fracture separations. Processes that can lead to these tensile zones are concentrated uplift resulting in a convex surface or dipping beds as seen in hogbacks that can slip down dip.

Quality assurance – A planned system of activities that provides the owner and permitting agency assurance that the facility was constructed as specified in the design. Construction quality assurance includes inspections, verifications, audits, and evaluations of materials and workmanship necessary to determine and document the quality of the constructed facility. Quality assurance refers to measures taken by the construction quality assurance organization to assess if the installer or contractor is in compliance with the plans and specifications for a project. An example of a quality assurance activity is verifications of quality control tests performed by the contractor using independent equipment and methods.

Quality control – A planned system of inspections that is used to directly monitor and control the quality of a construction project. Construction quality control is normally performed by the contractor and is necessary to achieve quality in the constructed system. Construction quality control refers to measures taken by the contractor to determine compliance with the requirements for materials and workmanship as stated in the plans and specifications for the project. An example of a quality control activity is the testing performed on compacted earthfill to measure the dry density and water content. By comparing measured values to the specifications for these values based on the design, the quality of the earthfill is controlled.

Re-filling – The procedure of filling a reservoir after it has previously held water, typically after a modification to an existing dam.

Re-grading – The mathematical procedure of removing a certain fraction of an original gradation, such as removing all gravel sizes (regrading on the No. 4 sieve size).

Relative density – A numerical expression that defines the relative denseness of a cohesionless soil. The expression is based on comparing the density of a soil mass at a given condition to extreme values of density determined by standard tests that describe the minimum and maximum index densities of the soil. Relative density is the ratio, expressed as a percentage, of the difference between the maximum index void ratio and any given void ratio of a cohesionless, free-draining soil to the difference between its maximum and minimum index void ratios.

Relief well – A vertical well at the downstream toe of the dam used to relieve pressure in a deeper foundation layer that is under high pressure.

Repair – The reconstruction or restoration of any part of an existing structure for the purpose of its maintenance.

Replacement – The removal of existing materials that can no longer perform their intended function and installation of a suitable substitute.

Reservoir (FEMA 2004) – A body of water impounded by an embankment dam and in which water can be stored.

Reservoir evacuation – The release or draining of a reservoir through an outlet works, spillway, or other feature at an embankment dam.

Riprap (FEMA 2004) – A layer of large, uncoursed stone, precast blocks, bags of cement, or other suitable material generally placed on the slope of an embankment or along a watercourse as protection against wave action, erosion, or scour. Riprap is usually placed by dumping or other mechanical methods and, in some cases, is hand placed. It consists of pieces of relatively large size as distinguished from a gravel blanket. *Rock fragments, rock, or boulders placed on the upstream or downstream faces of embankment dams to provide protection from erosion caused by wind or wave action.*

Risk (FEMA 2004) – A measure of the likelihood and severity of adverse consequences (National Research Council 1983). Risk is estimated by the mathematical expectation of the consequences of an adverse event occurring (i.e., the product of the probability of occurrence and the consequence) or alternatively, by the triplet of scenario, probability of occurrence, and the consequence.

Risk reduction analysis – An analysis that examines alternatives for their impact on the baseline risk. This type of analysis is begun once the baseline risk indicates risks are considered too high and that some steps are necessary to reduce risk.

Rock – Lithified or indurated crystalline or noncrystalline materials. Rock is encountered in masses and as large fragments, which have consequences to design and construction differing from those of soil.

Rockfill dam – See Dam, rockfill.

Roller – Machinery used to increase the density of soil that typically rolls across the fill on a drum. Also see Compactor.

Riprap bedding – The bedding layer under riprap usually consisting of gravel or cobble size material. The purpose of the bedding is to provide a transition between the riprap and upstream shell or core of the dam as the case may be.

Rutting – The tire or equipment impressions in the surface of a compacted fill that result from repeated passes of the equipment over the compacted fill when the soil is at a moisture and density condition that allows the rutting to occur. Rutting usually occurs when soils are not well compacted and/or are at a water content too high for effective compaction.

Sand (ASTM D653 2002) – Particles of rock that will pass the No. 4 (4.75-µm) sieve and be retained on the No. 200 (0.075-mm) U.S. standard sieve.

Sand boil – Sand or silt grains deposited by seepage discharging at the ground surface without a filter to block the soil movement. The sand boil may have the shape of a volcano cone with flat to steeper slopes, depending on the size and gradation of particles being piped. Sand boils are evidence of piping occurring in the foundation of embankments or levees from excessive hydraulic gradient at the point of discharge. Seepage emerging downstream of a dam, characterized by a boiling action at the surface and typically surrounded by a ring of material (caused by deposition of foundation and/or embankment material carried by the seepage flow).

Scour – The loss of material occurring at an erosional surface where a concentrated flow is located, such as a crack through a dam or the dam/foundation contact. Continued flow causes the erosion to progress, creating a larger and larger eroded area.

Second stage – The second stage of a filter/drain system usually consisting of gravel. The second stage protects the first stage and surrounds the drainpipe in toe drain systems.

Secondary defensive elements – Embankment zones whose purpose is to protect the core and foundation if an unexpected defect or condition presents itself. Also see Filter.

Seepage (ASTM D653 2002) – The infiltration or percolation of water through rock or soil or from the surface.

Segregation – The tendency of particles of the same size in a given mass of aggregate to gather together whenever the material is being loaded, transported, or otherwise disturbed. Segregation of filters can cause pockets of coarse and fine zones that may not be filter compatible with the material being protected.

Seismic activity – The result of the earth's tectonic movement.

Self-healing – The property of a soil in which soil particles rearrange themselves until they are stable. Also rearrangement of base soil particles against the face of a filter.

Settlement (FEMA 2004) – The vertical downward movement of a structure or its foundation.

Shear strength (ASCE 2000) – The ability of a material to resist forces tending to cause movement along an interior planer surface.

Shear stress – Stress acting parallel to the surface of the plane being considered.

Shell – In a zoned embankment, a shell zone typically is provided downstream of the core of the embankment, and may be provided upstream of the core as well, to provide stability to the dam embankment. Shell zones typically have significantly higher permeability than the core.

Silt (ASTM D653 2002) – Material passing the No. 200 (75-µm) U.S. standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air dried.

Single-stage filter – A filter consisting of a single material, usually used in reference to types of toe drains.

Sinkhole – A depression, indicating subsurface settlement or particle movement, typically having clearly defined boundaries with a sharp offset. A steep-sided depression formed when removal of subsurface embankment or foundation material causes overlying material to collapse into the resulting void.

Slaking – Degradation of excavated foundation caused by exposure to air and moisture.

Slope (FEMA 2004) – Inclination from the horizontal. Sometimes referred to as batter when measured from vertical.

Slotted pipe – See Perforated pipe.

Slough – See Slump.

Slump – Movement of a soil mass downward along a slope.

Slurry – A mixture of solids and liquids.

Soil – An earth material consisting of three components: solids (mineral particles), liquids (usually water), and gasses (air).

Soil resistivity – The measure of the resistance to current flow in a soil.

Soluble salt – A salt that can be dissolved in water.

Specifications – The written requirements for materials, equipment, construction systems, and standards.

Spillway – A structure that passes floodflows in a manner that protects the structural integrity of the dam. Where more than one spillway is present at a dam, the service spillway begins flowing first, followed by the auxiliary spillway (if three spillways are present), and finally the emergency spillway.

Stability (ASCE 2000) – The resistance to sliding, overturning, or collapsing.

Standard Proctor compaction test – A standard laboratory or field test procedure performed on soil to measure the maximum dry density and optimum water content of the soil. The test uses standard energy and methods specified in ASTM Standard Test Method D 698.

Standards (ASCE 2000) – Commonly used and accepted as an authority.

Static stability – The stability of a structure under normal operating conditions (as opposed to unusual loadings such as floods or earthquakes). Stability is typically evaluated as a factor of safety against sliding, overturning, or slope failure.

Storage (FEMA 2004) – The retention of water or delay of runoff either by planned operation, as in a reservoir, or by temporary filling of overflow areas, as in the progression of a flood wave through a natural stream channel.

Strip outlet drains – Drainage material placed in strips perpendicular to the dam axis under the downstream shell used to connect the base of the chimney with the downstream toe.

Structural height – The vertical distance from the lowest point of the excavated foundation (excluding narrow fault zones) to the top of the dam.

Subsidence – A depression, indicating subsurface settlement or particle movement, typically not having clearly defined boundaries.

Suffosion – Seepage flow through a material that causes part of the finer grained portions of the soil matrix to be carried through the coarser grained portion of the matrix. This type of internal erosion is specifically relegated only to gap-graded soils (internally unstable soils) or to soils with an overall smooth gradation curve, but with an overabundance of the finer portions of the curve represented by a "flat tail" to the gradation curve. While a crack is not needed to initiate this type of internal erosion, a concentration of flow in a portion of the soil is needed.

Surficial deposits – The relatively younger materials occurring at or near the Earth's surface overlying bedrock. They occur as two major classes: (1) deposits generally derived from bedrock materials that have been transported by water, wind, ice, gravity, and man's intervention and (2) residual deposits formed in place as a result of weathering processes. Surficial deposits may be stratified or unstratified, and may be partially indurated or cemented by silicates, oxides, carbonates, or other chemicals (caliche or hardpan).

Tailings – The fine-grained waste materials from an ore-processing operation.

Tailings dam – See Dam, tailings.

Tailwater (ASCE 2000) – The elevation of the free water surface (if any) on the downstream side of an embankment dam.

Toe drain – A drain typically located at the downstream toe of a dam although drains under the downstream shell and downstream of the toe of the dam are also considered toe drains. The purposed of the drain is to gather flow from the chimney and blanket if provided and to collect seepage from the foundation. Toe drains can be either be single-stage or two-stage filter/drain systems and may or may not include a collection pipe. Open-jointed tile or perforated pipe located at or near the toe of the dam that functions to collect seepage and convey the seepage to a downstream outfall.

Toe of the embankment dam (FEMA 2004) – The junction of the downstream slope or face of a dam with the ground surface; also referred to as the downstream toe. The junction of the upstream slope with ground surface is called the heel or the upstream toe.

Transition zone – A zone in an embankment dam that provides a transition in grain size between two zones that are not filter compatible.

That is one zone does not meet the particle retention criteria for the other. An example of a transition zone would be a zone required between a clayey gravel core and a downstream cobble shell.

Transverse crack – A crack that extends in an upstream and downstream direction within an embankment dam.

Trench – A narrow excavation (in relation to its length) made below the surface of the ground.

Trip – The single movement of a piece of compaction equipment from beginning to end of a section of material being compacted. See also "Passes."

Two-stage filter – A filter consisting of two materials, usually used in reference to types of toe drains. The materials are typically a sand filter used to protect the foundation and a gravel drain used as the transition around a perforated collector pipe. In this example the filter would also be known as stage 1 and the gravel as stage 2.

Tunnel (FEMA 2004) – A long underground excavation with two or more openings to the surface, usually having a uniform cross section, used for access, conveying flows, etc.

Turbidity meter (ASCE 2000) – A device that measures the loss of a light beam as it passes through a solution with particles large enough to scatter the light.

Uniform gradation or uniformly graded – A soil gradation consisting primarily of one size.

Unwater – Removal of surface water; removal of visible water; removal of water from within a conduit.

Uplift (ASCE 2000) – The pressure in the upward direction against the bottom of a structure such as an embankment dam or conduit.

Upstream blanket – An impervious soil layer placed upstream of the dam and connected to the core. The purpose of an upstream blanket is to increase the seepage path length under the dam on pervious foundations.

Vertical filter – A zone in an embankment dam near the embankment midsection which has a vertical side slopes. Also known as a chimney or chimney filter.

Void – A hole or cavity within the foundation or within the embankment materials surrounding a conduit.

Water content (ASTM D653 2002) – The ratio of the mass of water contained in the pore spaces of soil or rock material, to the solid mass of particles in that material, expressed as a percentage.

Weir (ASCE 2000) – A barrier in a waterway, over which water flows, serving to regulate the water level or measure flow. A device designed to allow the accurate measurement of the flow rate of drain flows, seepage flows, etc., by forcing the water to flow through a standardized opening, and measuring the elevation differential between the water surface in the stilling pool in front of the weir and the weir crest elevation, using a staff gauge set back an appropriate distance from the weir. When a weir is installed in a standard manner, charts are available for correlating staff gauge readings with flow rates. Types of weirs include Cipolletti, rectangular, and V-notch.

Well-graded – A soil gradation consisting of several soil sizes.

Zone – An area or portion of an embankment dam constructed using similar materials and similar construction and compaction methods throughout.

Attachment A – Base Soil Selection

Introduction

As defined in this manual, the base soil is the soil being protected by a filter. For protective filters, the flow of water is from the base soil towards and into the filter. The base soil can be naturally occurring deposits (in situ deposits) or earthfill placed during construction. For toe drains and filter blankets, the base soils are usually naturally occurring deposits since these filters are placed against natural or excavated surfaces. Chimney filters are placed against earthfill as part of original construction or existing embankment zones during embankment dam modifications.

Base soil variability

Understanding variability of the base soil is instrumental in designing adequate filter protection. While there will always be variability in base soils, typically there is greater variability in natural soil deposits than earthfill materials. Earthfill materials will have greater uniformity due to the mixing that occurs during excavation and placement operations. This is illustrated in Figures A-1 and A-2. Figure A-1 is a gradation plot of seven

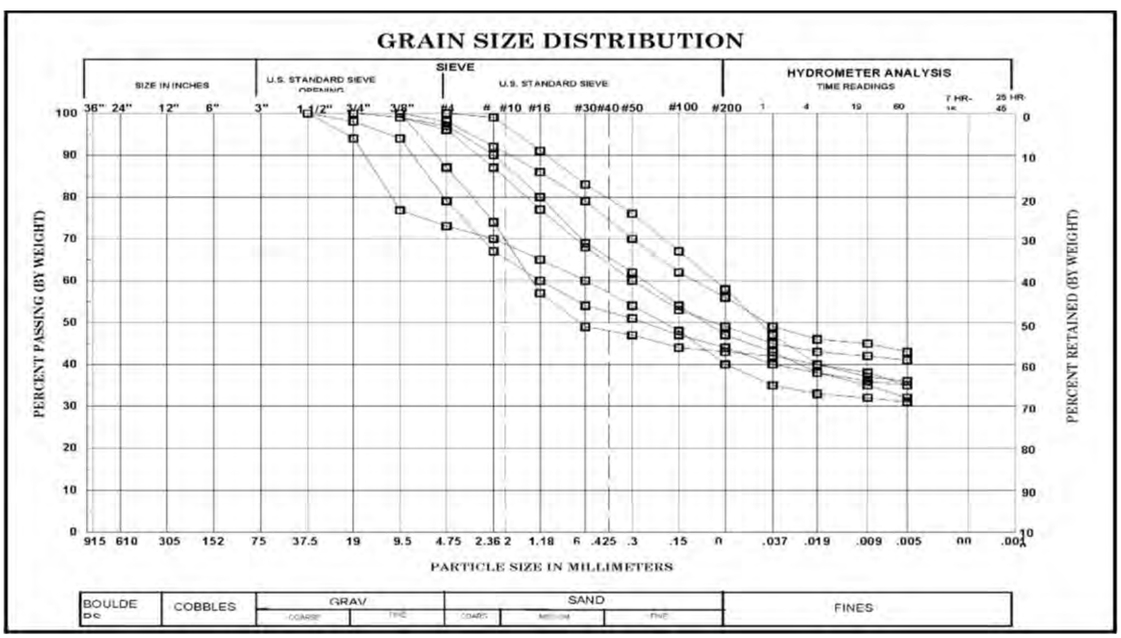

Figure A-1. Gradation plot of example core material.

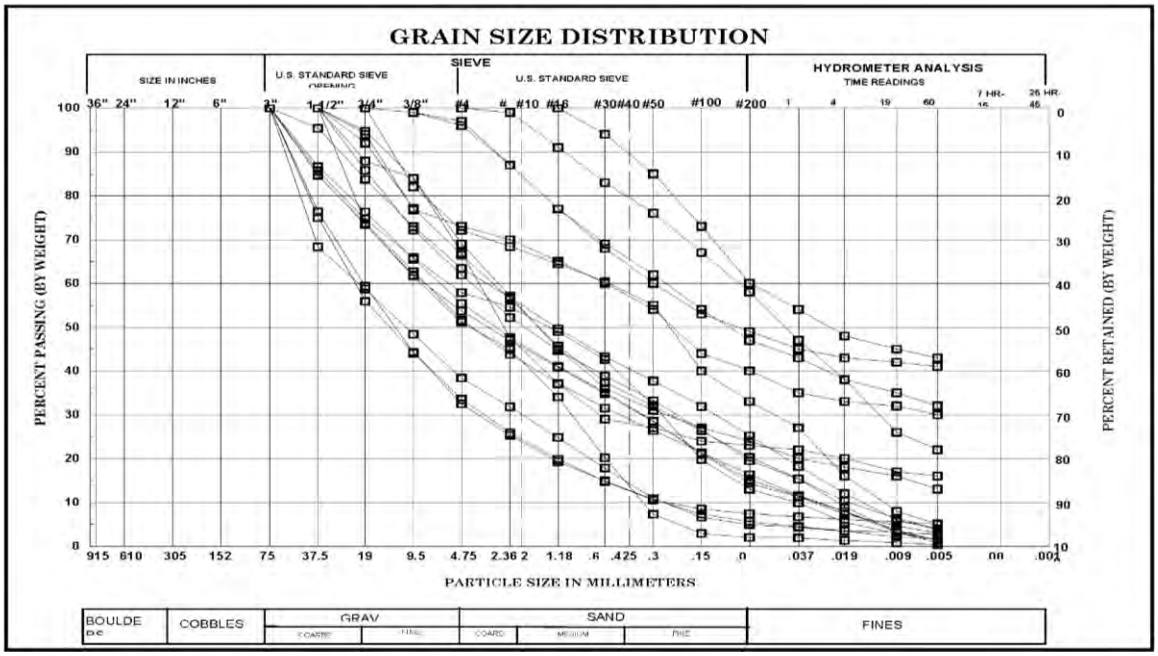

Figure A-2. Gradation plot of example foundation materials.

samples of core material from an existing dam, and Figure A-2 is a gradation plot of 19 samples of the foundation material for that dam. The figures illustrate one of the first questions that must be answered when performing filter design, "What is the soils category?" For the core material of this example, all samples are classified as Category 2 (40 to 85% fines), whereas the foundation samples classify into Categories 2, 3, and 4. Since the filter design procedure is based on designing for a single category, the question is raised, "Which category should be used?"

While the previous paragraph addressed core material found at existing dams, consideration for new construction is slightly different. Figure A-3 illustrates soil gradations taken from samples obtained from a borrow area intended for use as impervious core material. Recognizing the uniformity of this borrow area, an average gradation of the samples can be found.[1]

---

[1] Note the average gradation is calculated on the vertical axis of a gradient plot as shown in Figure A-17 at the end of this attachment. "Averaging" on the horizontal (logarithmic) scale will cause erroneous results.

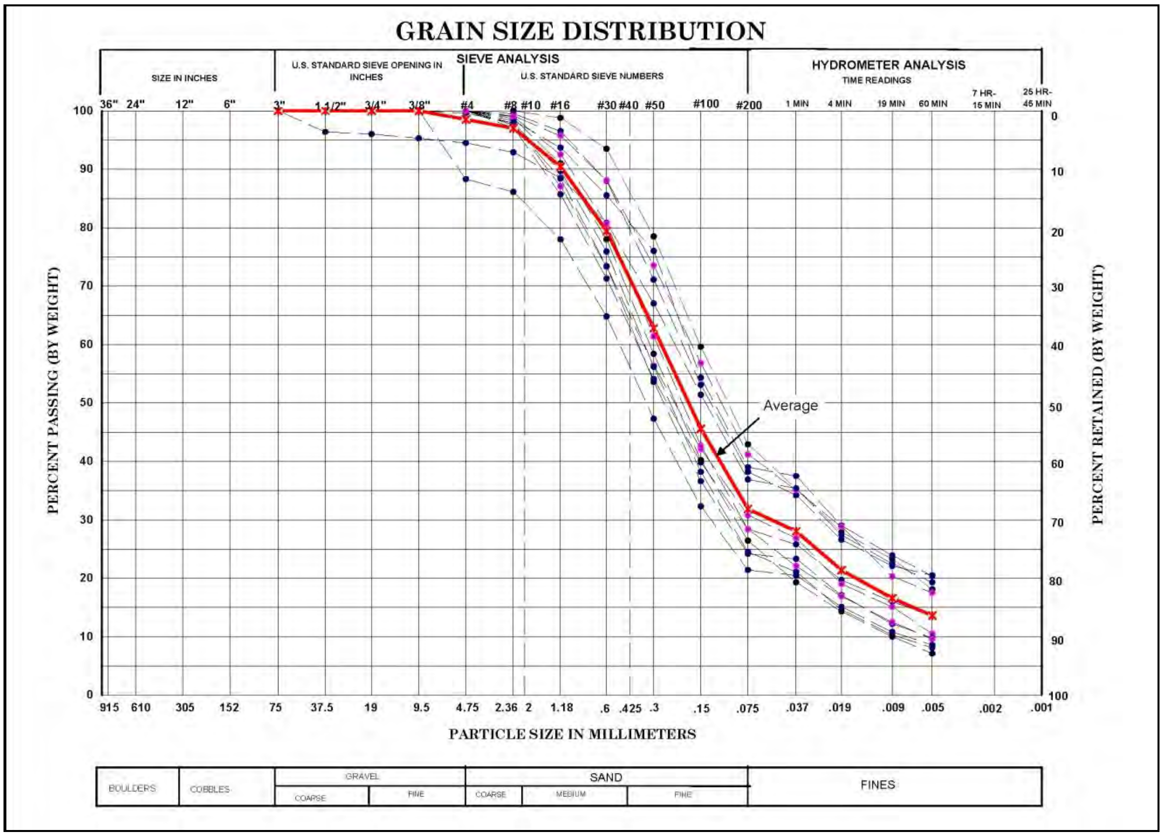

Figure A-3. Gradation plot of samples taken from a potential borrow source for a core material with little variability.

The average results as a single gradation can then be used for filter design. While using a single gradation to represent a material simplifies the filter design process, it can lead to problems that are described later. In a similar manner, the assumption that the finer side boundary of a band of gradations can act as a single conservative representation of that band can also lead to difficulties. Use of an "average" gradation should only be used when the borrow source exhibits uniformity and sufficient exploration has been performed to substantiate that assumption. Designing from the finer side of the band is described in more detail in the "Representative Base Soil Selection Procedure" section.

Geologic interpretation

As described in Chapter 5, base soils are categorized according to their fines content. Subsequent design calculations are dependent on this categorization, and incorrect categorization can result in an improperly designed filter. The incorrect categorization of soils can come from:

- Incorrect geologic interpretation
- Incorrect sampling
- Grouping two or more materially different soils into one geologic unit
- Inclusion of outliers in the gradations analyzed.

For naturally occurring deposits, difficulty arises in the categorization of the foundation units when the aerial extent of the units is small. Geologic categorization of foundation units is usually dependent of the geologic process that led to deposition. That is, the foundation strata may be differentiated into “alluvium,” soil deposited by swift moving water, and “aeolian,” soil deposited by wind. Note that this type of categorization is not dependent on the physical properties of the soil although, typically, the physical properties almost always vary based on depositional process. In this instance, different filters can be designed for each unit, when the stratigraphy is well understood. In some instances, foundations may include geologic units that are subsets of one geologic process, such as several alluvial units (alluvium 1, alluvium 2, and alluvium 3). In this case, all three units can be combined into a single alluvium unit for the purpose of base soil characterization. This situation is illustrated in Figure A-4, which shows the results of gradation tests on 10 soil samples. The original geologic cross section, shown in Figure A-5, indicates three alluvial subunits, Qal1, Qal2, and Qal3.

Examination of the gradation indicates that the three subunits are not different, based on grain size distribution, since none of the units can be grouped together in a distinct band. Therefore, for the purpose of filter design, the three units can be grouped together into one material, alluvium, as shown in Figure A-5.

The converse of the previous situation can also be true—geologic classification has grouped together two soils that have different grain size distributions. Figure A-6 illustrates a cross section through an alluvial fan that has been mapped as one geologic unit. Figure A-7 includes the gradation plots for the 19 samples taken in the alluvial fan and illustrates that two distinct groupings exist within the samples, Base 1 and Base 2. The Base 1 gradations are Category 2 soils, whereas the Base 2 soils are Category 3 and 4.

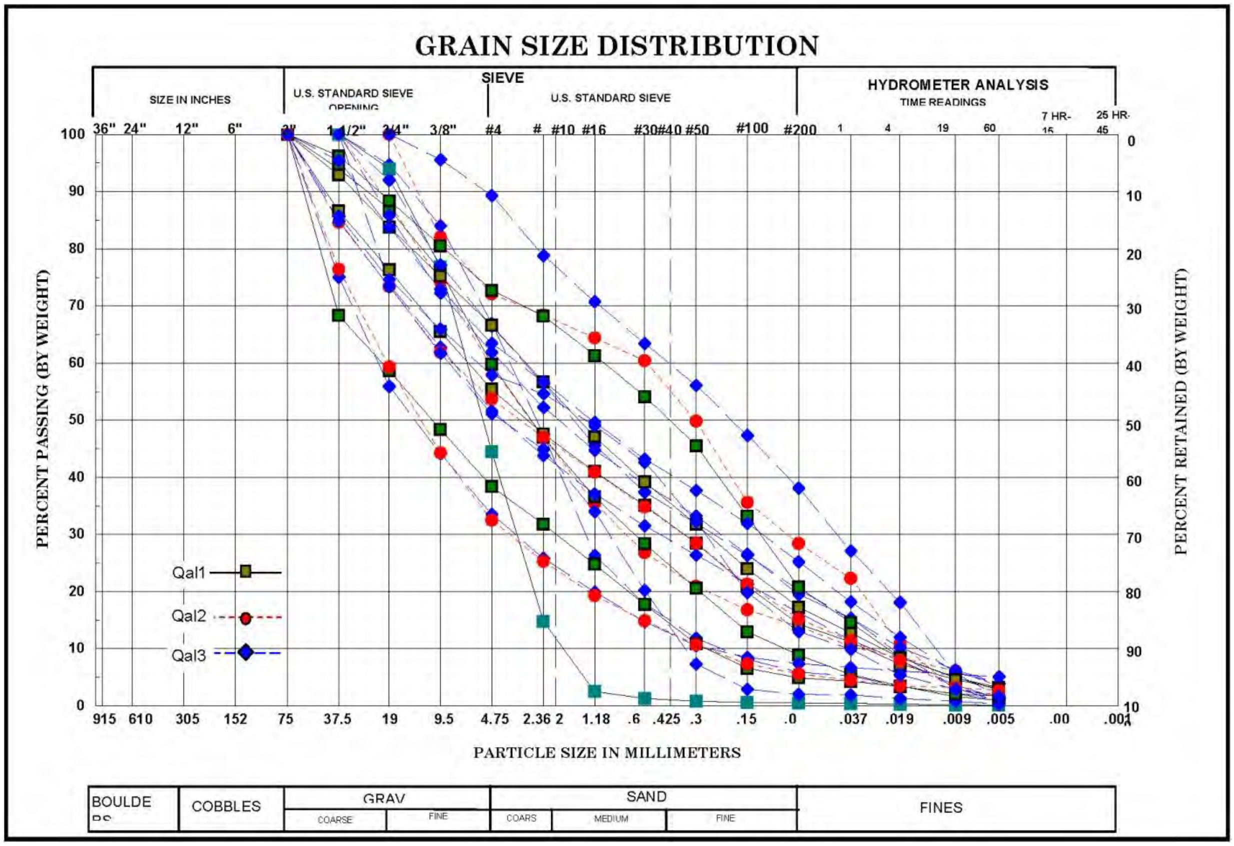

Figure A-4. Gradation plots of three alluvial deposits.

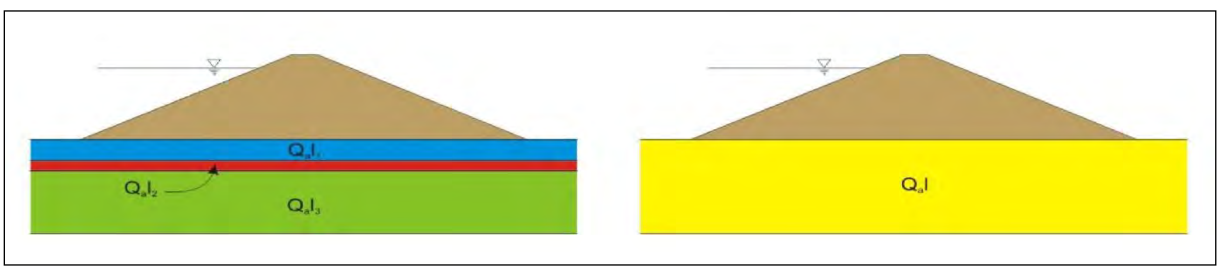

Figure A-5. Geologic cross section of three alluvial deposits that is simplified to one unit due to material uniformity.

By going back to the drill logs, it is seen that the six samples of Base 1 are in the upper portion of the fan and the 13 samples of Base 2 are in the lower portion. Therefore, the alluvial fan should be separated into two subunits for filter design, Qf1 and Qf2. If the two bases were not separated, the filter design procedure would result in a filter gradation for the Base 1 that would likely act as a barrier to the Base 2 soils. The barrier issue is described in more detail in the “Filter Barrier” section. Also note that if the samples were randomly distributed through the fan, a separation could not be made.

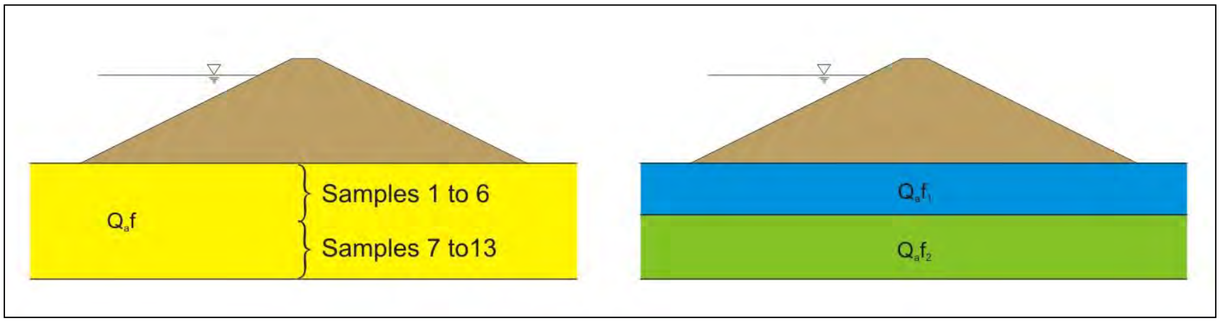

Figure A-6. Geologic cross section of a single alluvial fan deposit that is separated into two distinct units due to differences in material gradation.

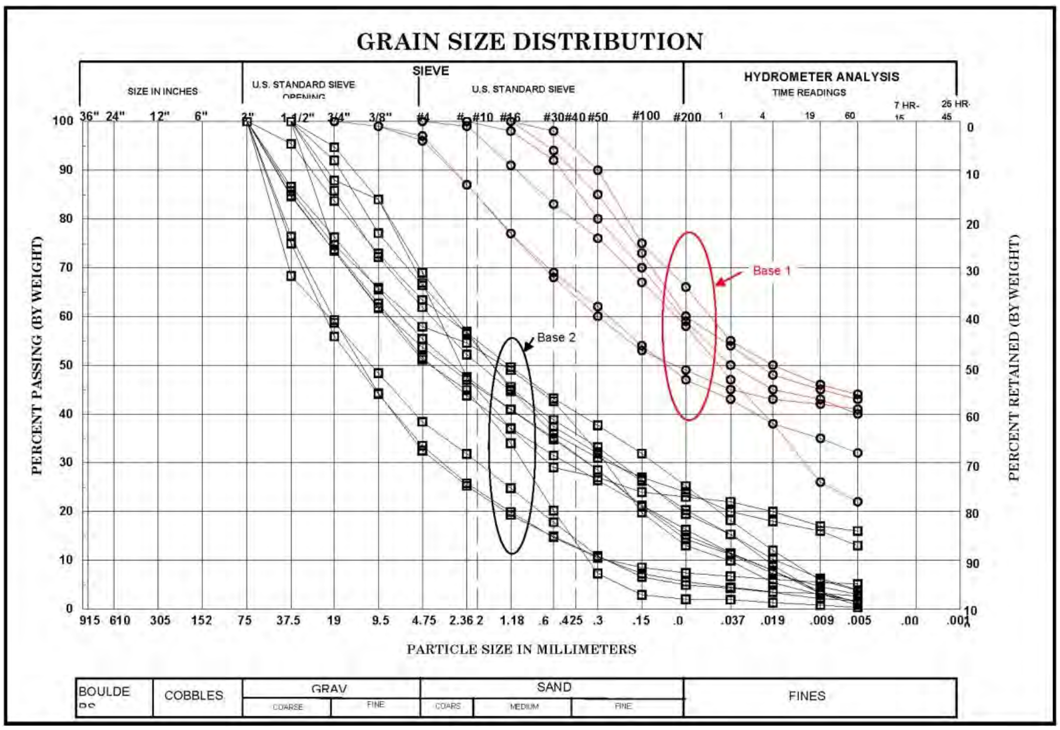

Figure A-7. Gradation plot of alluvial fan material indicating two distinct units.

Undifferentiated units

Undifferentiated units are more difficult to classify for use in filter analysis. Figure A-8 illustrates complex layering of silts, sands, gravels, and combinations of each. Alluvial processes can lead to these types of deposits due to the wide variety of depositional energy provided by river systems. Most erosion and depositional processes in river valleys occur during flood events. Near the rivers thalweg, or the deepest portion of the channel, the energy is the highest and the largest particles are moved. Further from the thalweg, the energy is not as great and only smaller particles are moved or

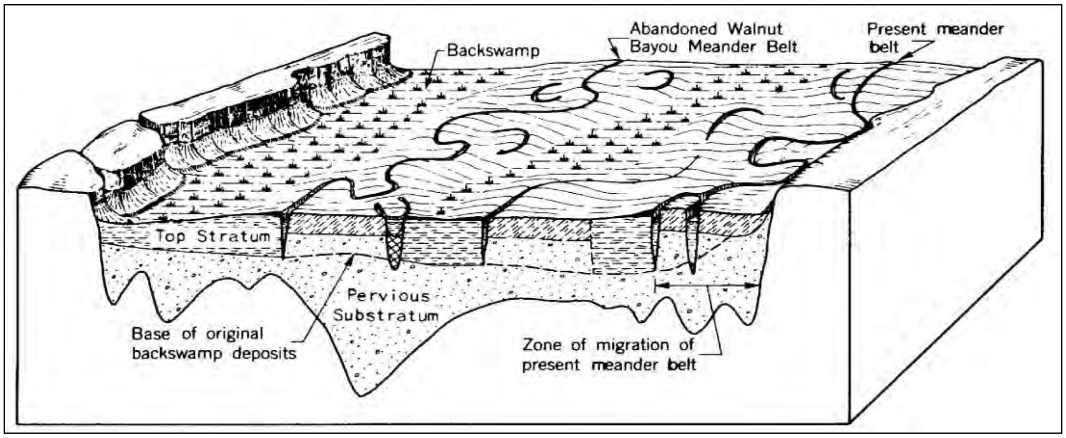

Figure A-8. Meandering pattern of Mississippi River near Vicksburg, Mississippi, illustrating how a variety of materials can be deposited across a valley as the river changes course over time.

the energy is so low finer material is deposited. It cannot be assumed though that the current location of the thalweg will indicate the location of coarsest deposits. Since the central water course of a river will work its way back and forth across the valley (meander), coarser material can be found anywhere. Commonly "buried channels" or "abandoned channels" are identified during exploration and, unfortunately sometimes during construction, when sufficient exploration is not undertaken. The sinuous nature of riverflow also complicates the erosion and depositional process. Rivers flow in a sinuous or serpentine course through their valleys. The extent of this "S" shape flow is a function of the amount of energy that needs to be shed for the given grade. Through geologic time, this serpentine path will cut across itself over and over. These are the processes that lead to the convoluted depositional sequence illustrated in Figure A-8.

While the previous example describes the method by which widely varying deposits can occur in alluvium, similar deposits are also seen from glacial and alluvial fan processes.

It should be noted that extent and continuity are difficult to ascertain for undifferentiated deposits. One may conclude from drawing a simple upstream to downstream cross section that a unit of particular interest is not continuous since it is truncated by other materials. Consider the case where a gravel deposit is identified but the cross section shows that it is truncated by silts and clays. Since the gravel layer may actually have a serpentine alignment, it would be incorrect to assume it is truncated as shown in the cross section. This situation is illustrated in Figure A-9.

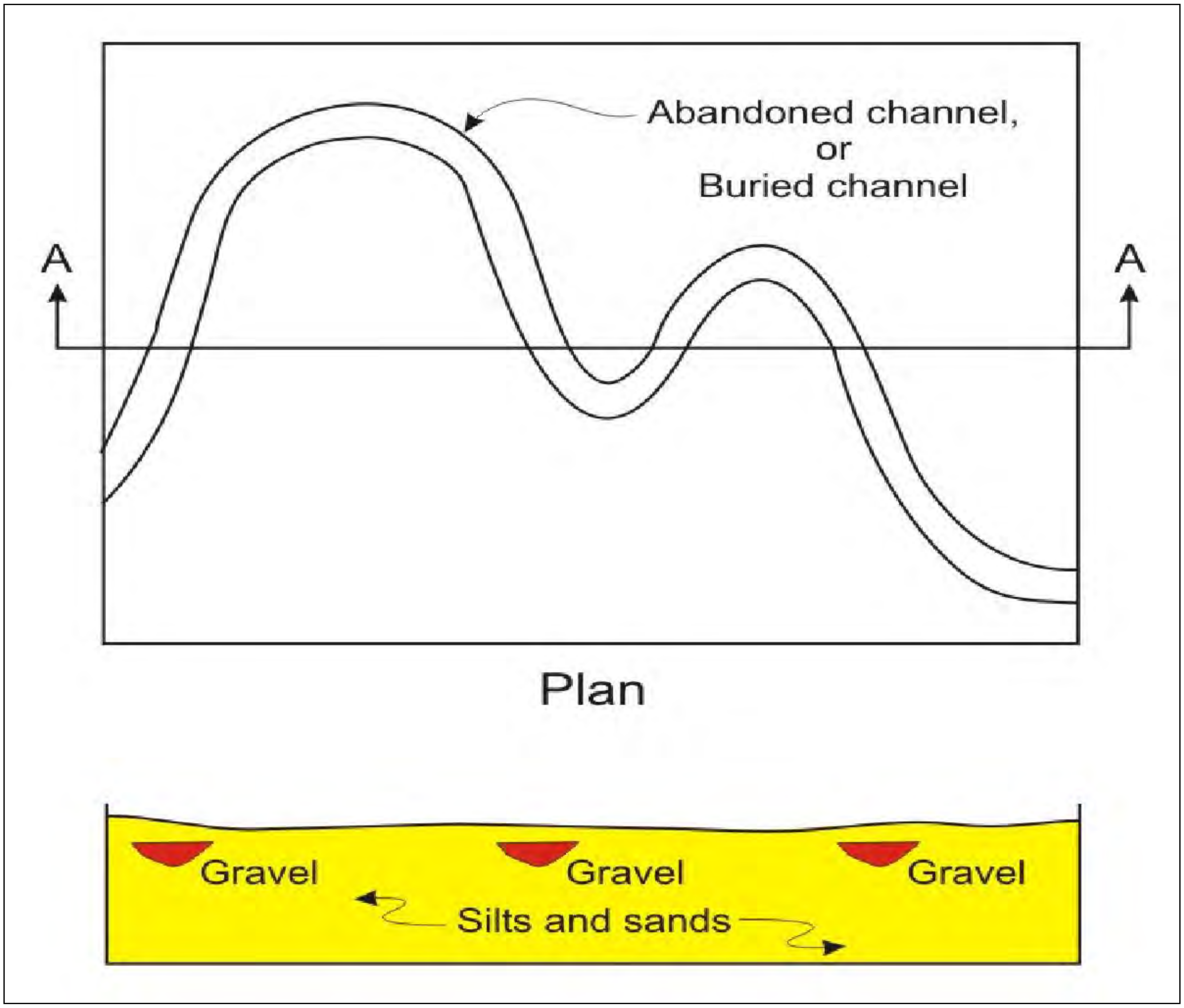

Figure A-9. Plan and sectional view of a meander illustrating so-called gravel lenses.

Outliers and sampling errors

Gradation test data may include statistical results that are not considered to represent in situ conditions. This is illustrated in the gradation plot of Figure A-10. Here, a single sample is more fine-grained than the seven other samples that all fall within the same gradation range. A number of factors could lead to this one sample being different from the other seven:

1. Incorrect sampling method (i.e., technician did not include larger test pit material because it would not fit in the bag or was too heavy)
2. Gradation test was performed incorrectly
3. Inventory error (i.e., sample is actually from another location)

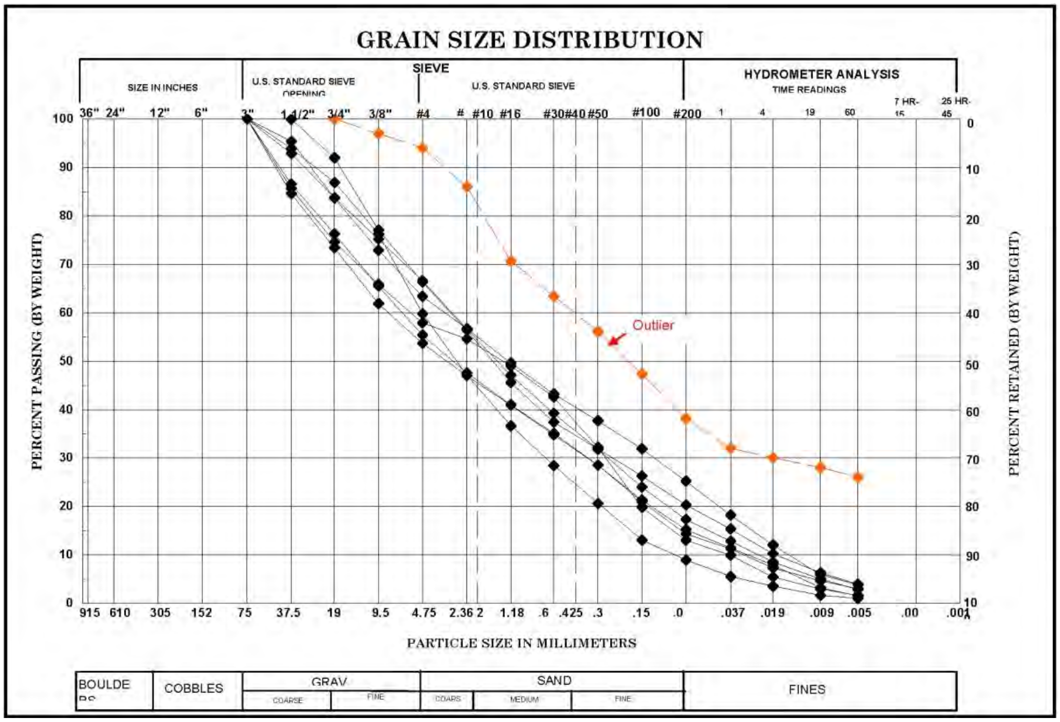

Figure A-10. Gradation test samples including an outlier.

4. Sample was taken from near the ground surface (topsoil), which will be removed during construction (stripping).

The designer should investigate these factors to ensure that the sample is valid for inclusion in the data set. If an error is found, it should be corrected so that accurate information is included in the data set. If it is found to be valid, outliers should be removed from the base soil test data since they do not represent foundation conditions. It is difficult to provide rules for exclusion of outliers, but they are generally identified visually as illustrated in Figure A-10. Eliminated outliers should not be greater than 15% of the sample set. If it is thought that greater than 15% of the sample set is outliers, the geologic interpretation, as described in "Geologic Interpretation" section should be studied.

Another error that can arise in categorizing borrow areas is related to sampling errors. One of the most common errors in this regard is the use of undersized samplers. The commonly used split spoon sampler has an inside opening size of 1-7/8 in., indicating that it is unable to sample coarse gravel and cobbles. Omission of these grain sizes can lead to incorrect base soil categorization and filter design, even with re-grading. Similar errors can occur with other, larger size samplers. The designer should always check that the correct size sampler is used for the expected exploration conditions. As described in Section 6.10.1, Identifying and Investigating Material Availability, the use of test pits is the preferred exploration method for evaluation of base soils. Collecting bag samples of materials obtained from these pits provides the most accurate base soil data as well as an indication of stratigraphy (layer) information that may not be detected from drillhole data.

Caution should also be exercised in how far a given sample is from a particular design element. As an example, consider an exploration program executed across a site in which samples are taken every 10 ft. In some drillholes, the first sample, at the 10-ft depth, could not be retrieved. Samples from the successful 10-ft depth, as well as, 20- and 30-ft depths were tested and used to represent the foundation soil (base soil). It is planned to construct a 6-ft-deep toe drain at the site using these data. It should be recognized that this exploration program did not address the upper 10 ft of the foundation and that layer could be materially different than what is seen lower. Therefore, this base soil could be misleading and result in an incorrectly designed filter for the toe drain.

Filter barriers

Using standard filter design procedures, it is possible to design a filter that is less permeable than portions of the foundation. Such a barrier is illustrated in Figure A-11. The figure represents a lenticular foundation of undifferentiated soil deposits. While no distinct layer of gravel is present, concentrated seepage can occur through the more pervious lenses. As

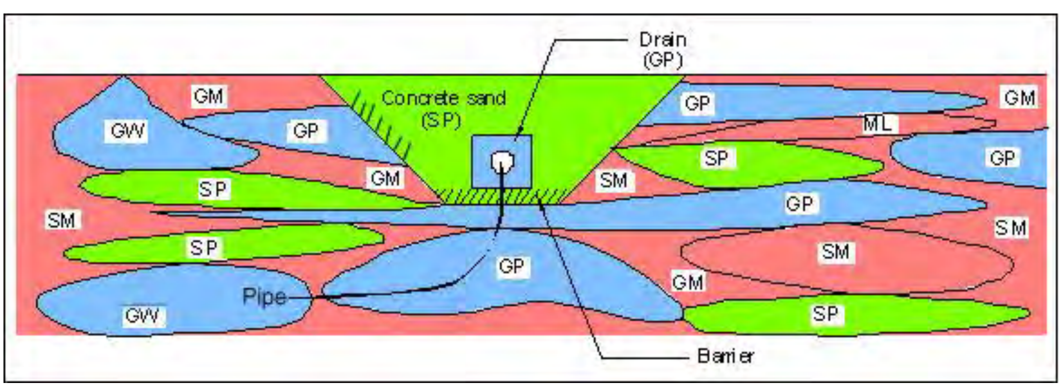

Figure A-11. A filter for a toe drain that is acting as a barrier to a more pervious foundation layer.

shown in the figure, a sand filter will then act as a barrier at the bottom of the trench. This can result in less than expected flow quantity entering the pipe and higher pressures.

Figure A-12 is a second method of visualizing this issue. This figure summarizes the base soil gradations as well as a proposed filter. The base soil is shown by the limits of the re-graded curves of the foundation soil samples. The re-grading consists of scalping (mathematically) the material larger than the No. 4 sieve as described in filter design procedures. Also shown on the plot is the average gradation for concrete sand, a common filter material. The hatched portion of the graph indicates the range of base soil gradations that would be coarser than the filter. Since this filter would be finer than these base soil gradations, it would act as a barrier to those materials (about 25% of the total base soil range taken at the D_{15} size).

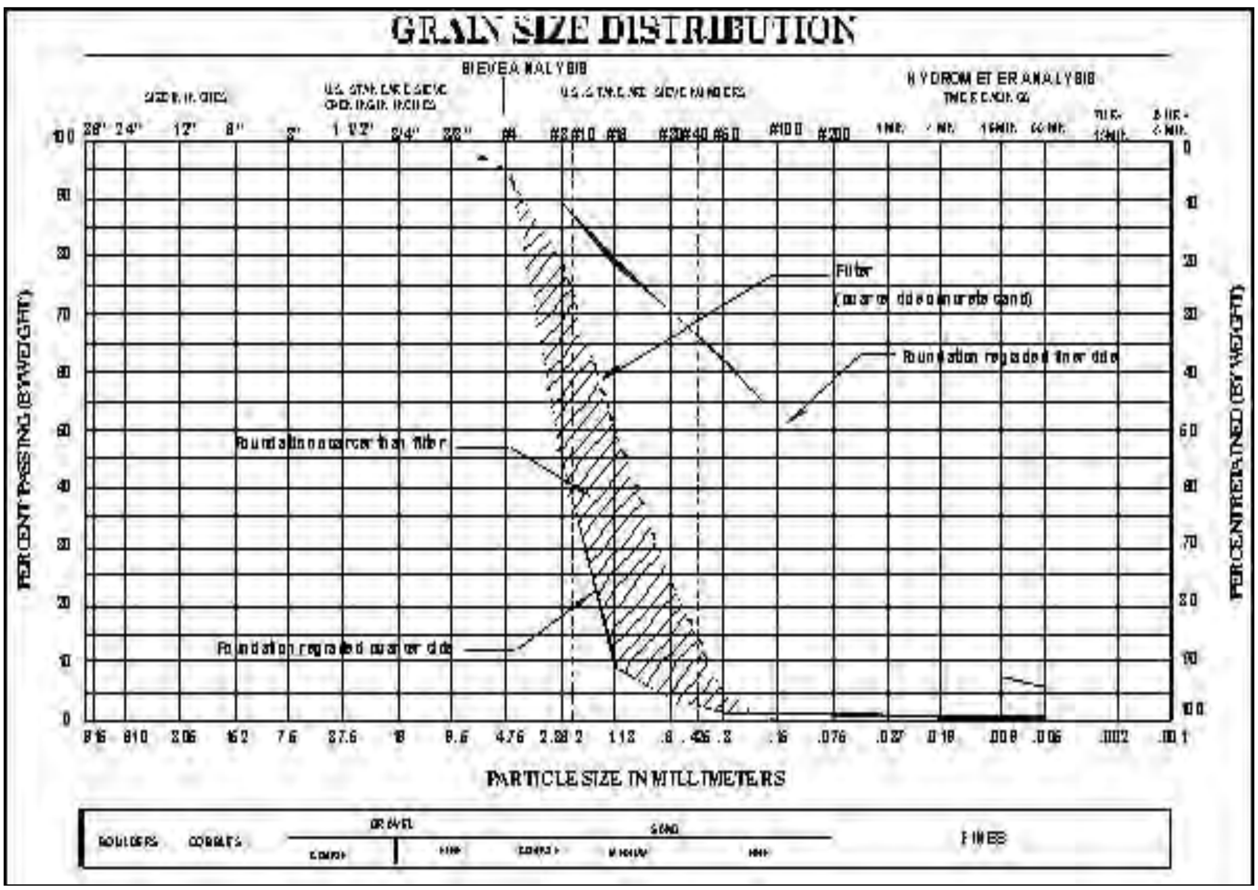

Figure A-12. The filter barrier concept illustrated on a grain size distribution plot.

Figure A-13 illustrates the effect of a filter barrier on theoretical soil deposits. The upper portion of the figure shows a box consisting of three layers of soil, each 1-ft thick. A head is applied to the left side of the box and drain on the right side. The configuration results in a head drop of 10 ft across the box. The box is 100 ft long, as is the flow length. Utilizing Darcy's equation, total flow through the box is calculated as indicated in the figure. The resultant total flow for this arrangement is 5×10^{-1} ft^3/min.

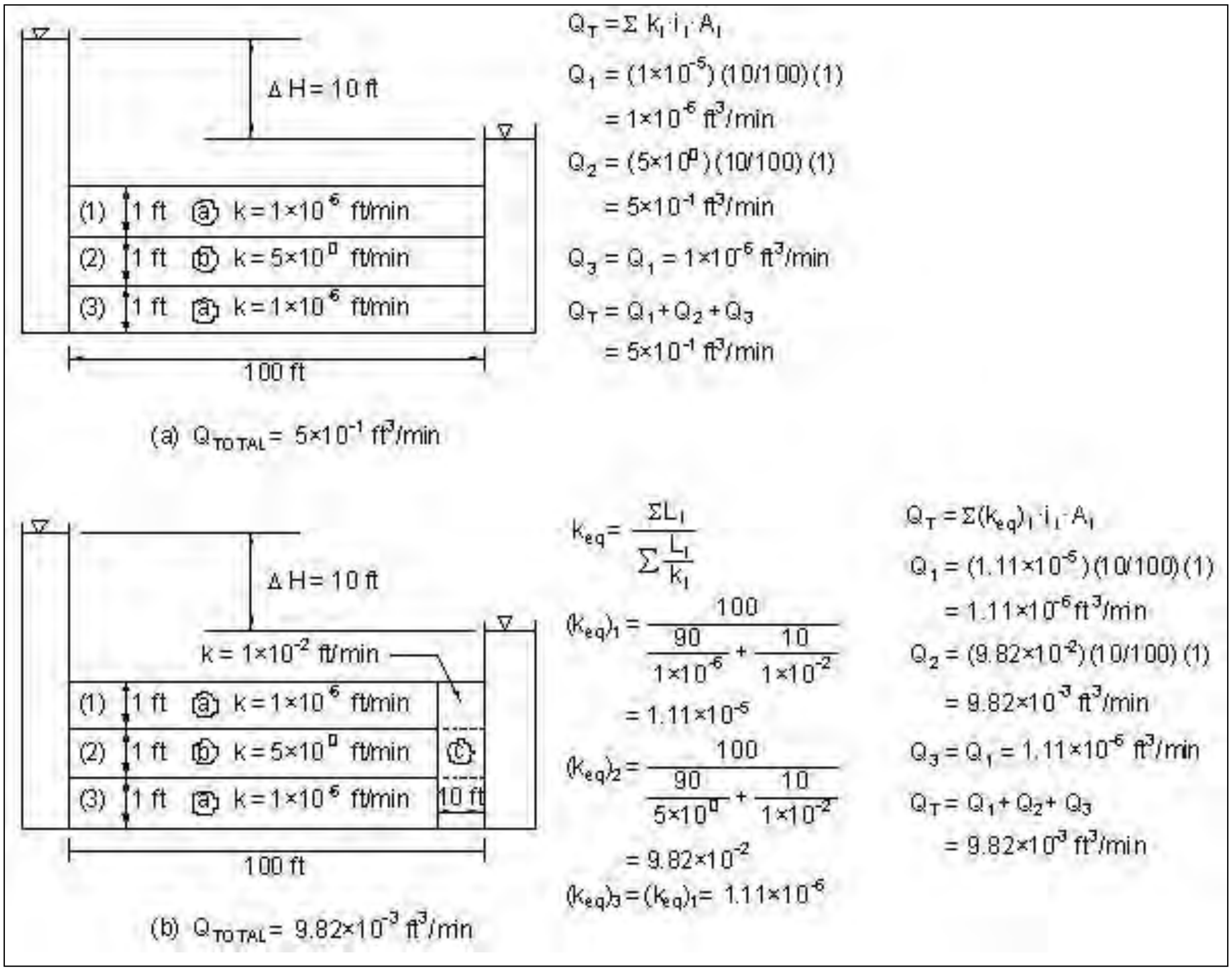

Figure A-13. The filter barrier concept illustrated as flow through a laboratory box.

Taking the same arrangement and adding a filter barrier, shown on the right side of the box in the lower figure, results in a total flow of 9.8×10^{-3} ft^3/min., 1/50th of the original flow.

Representative base soil selection procedure

Since the filter design procedure is based on the use of a single gradation, a procedure is required to transition from multiple base soil gradations. The process is further complicated if the base soil can be classified within two or more categories. This section will describe a procedure, as expressed in a flowchart, which addresses the issues discussed in the four preceding sections. Since considerations for earthfill and in situ soils are different, two procedures are used. The selection process for earthfill is shown in Figure A-14, and the process for in situ (foundation) soils is shown in Figure A-15.

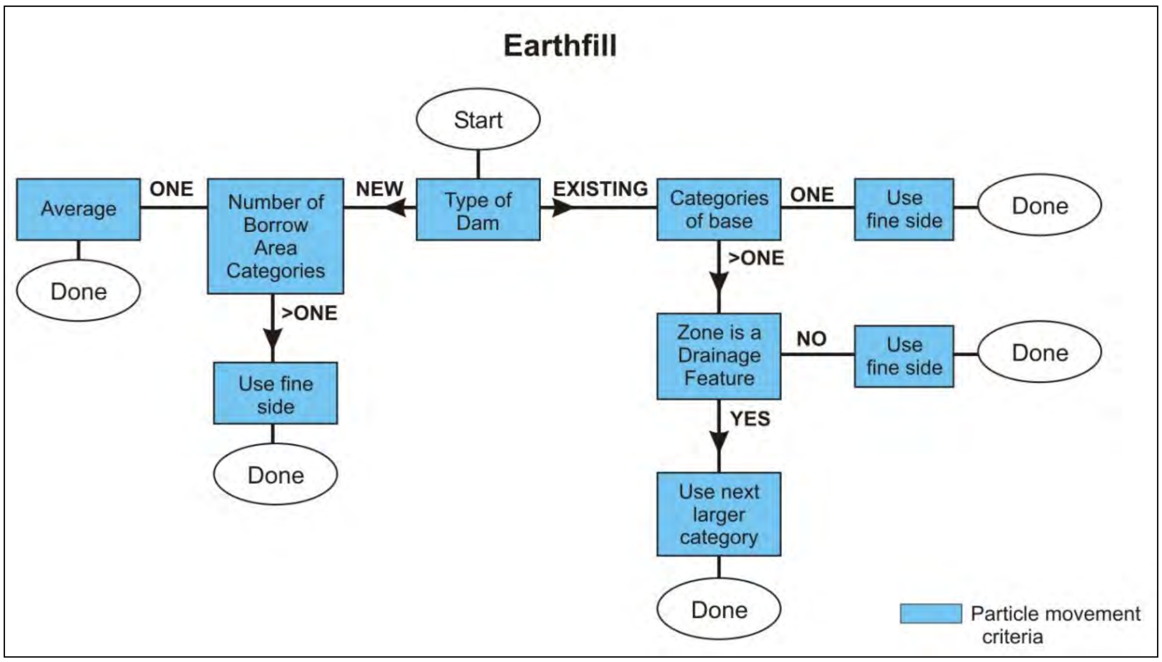

Figure A-14. Selection process for earthfill base soils.

As illustrated on Figure A-14,[1] the first step in base soil categorization is to determine if the dam is new or existing. For new dams, if the base soil falls within one category then the average gradation of the base soil samples is used (see Figure A-3). If the base soils fall within more than one category for new or existing dams, then use the finer side of the range of gradations. Note: the finer side of a range of gradations is illustrated in Figure A-16.

[1] Filter design in this flowchart is controlled by particle retention criteria.

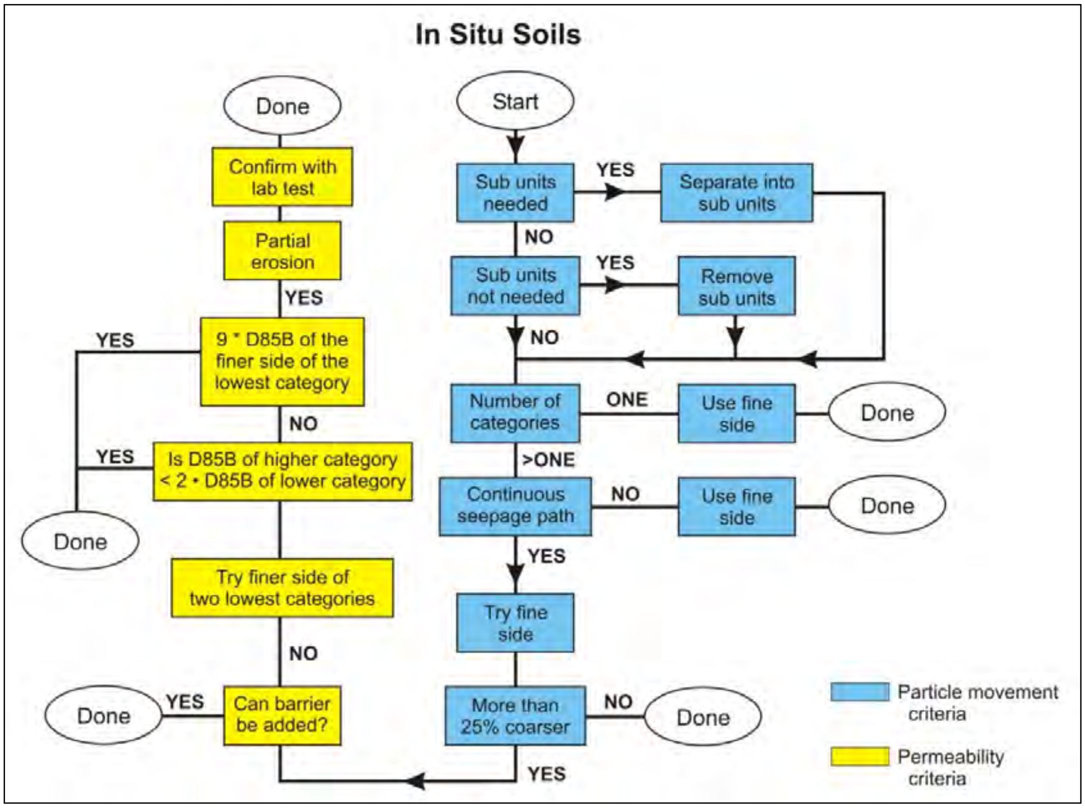

Figure A-15.—Selection process for in situ base soils. When applying permeability design criteria, use the coarse side base gradation when calculating the minimum allowable D5F (Step 6, Chap. 5).

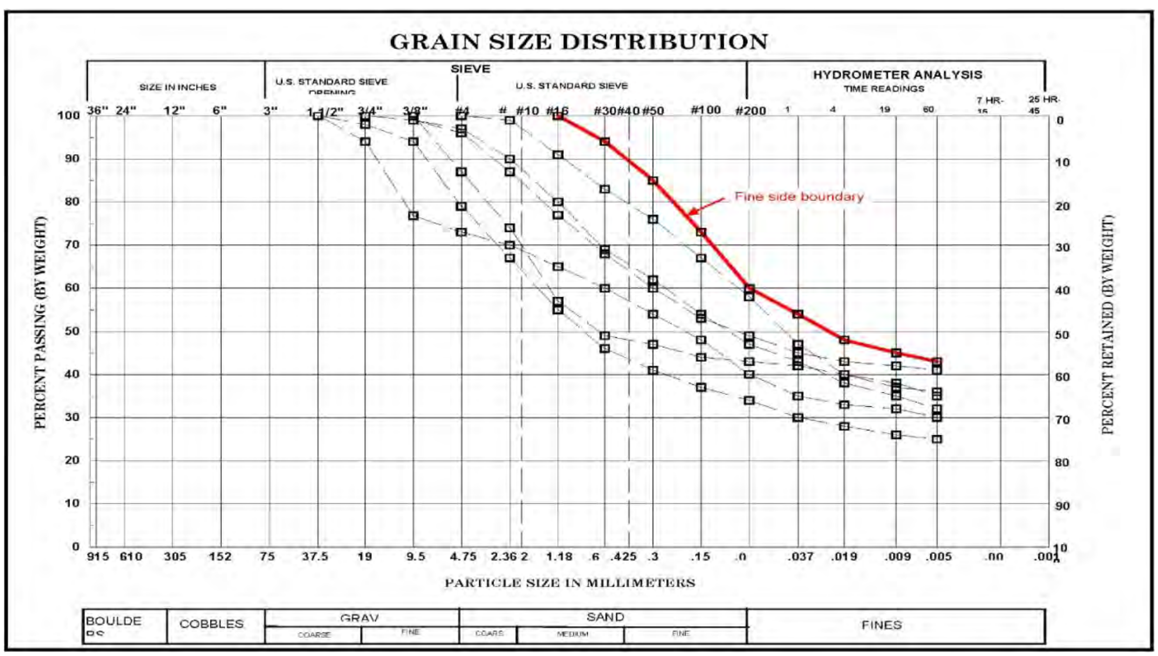

Figure A-16. Example of finer side of a range of soil gradations.

For an existing dam, if the earthfill materials fall within one category, the fine side of the range of gradation is used. If the earthfill falls within more than one category and it is not a drainage feature (toe drain, relief well, etc.), it too can be based on the finer side of the range of gradations. If an earthfill base is placed into more than one category, and it is used as a drainage feature, use the finer side of the next highest number category.

Base soil selection for in situ soils is more complicated due to the greater variability of natural soil deposits over earthfill. This selection process does not differentiate between existing and new dams since it is not germane. Using Figure A-15,[1] the first steps are to check whether the in situ materials are categorized correctly based on grain size distribution as described in the "Geologic Interpretation" section. After this is complete, determine how many categories the range of base soils fall within. If only one category is present, select the fine side of that category. If more than one is present, determine if a continuous seepage path is present, as described in the "Geologic Interpretation" section. If the seepage path is not continuous, use the finer side of the lowest number category. If a continuous seepage path is present, perform a trial design using the fine side of the lowest numbered category. Check if the finer side of the trial filter gradation is finer than 25% of the base soil gradations. If no more than 25% of the base soils are coarser than the fine side of the trial filter, the trial is acceptable. If more than 25% of the base soil gradations are coarser than the fine side of the filter, the overall project design should be evaluated. Design elements that reduce the volume of seepage that should be considered for this situation are cutoff walls, upstream blankets, and grouting.

If the design elements cannot be addressed, or site conditions are exceptionally poor (usually at existing dams), or costs are prohibitive, then the design proceeds by emphasizing permeability requirements instead of particle retention requirements. This is accomplished by comparing the trial filter design based on the finer side of the two lowest numbered categories. If the $D_{85}B$ of the higher numbered category is less than twice the $D_{85}B$ of the lower numbered category, the design based on the higher numbered

[1] Filter design in this flowchart is controlled by particle retention criteria for some cases and permeability for other cases. The different cases are described in the narrative.

category is acceptable. Note that this design eliminates the factor of safety against particle movement that is implicit in all designs that meet particle retention criteria.

If the $D_{85}B$ of the higher numbered category soil is more than twice the $D_{85}B$ of the lower numbered category, perform a new trial. In that trial, find the $D_{15}F$ of the filter by multiplying the $D_{85}B$ of the finer side of all gradations by 9. That is:

$$D_{15}F = 9 * D_{85}B$$

Additionally, recompute the minimum allowable $D_{15}F$ (Step 6, Chapter 5) using the coarser side base gradation. This will result in a filter that will allow partial, but not continuous, erosion. This design should always be confirmed by a laboratory filter test using the lowest category soil and the proposed filter material.

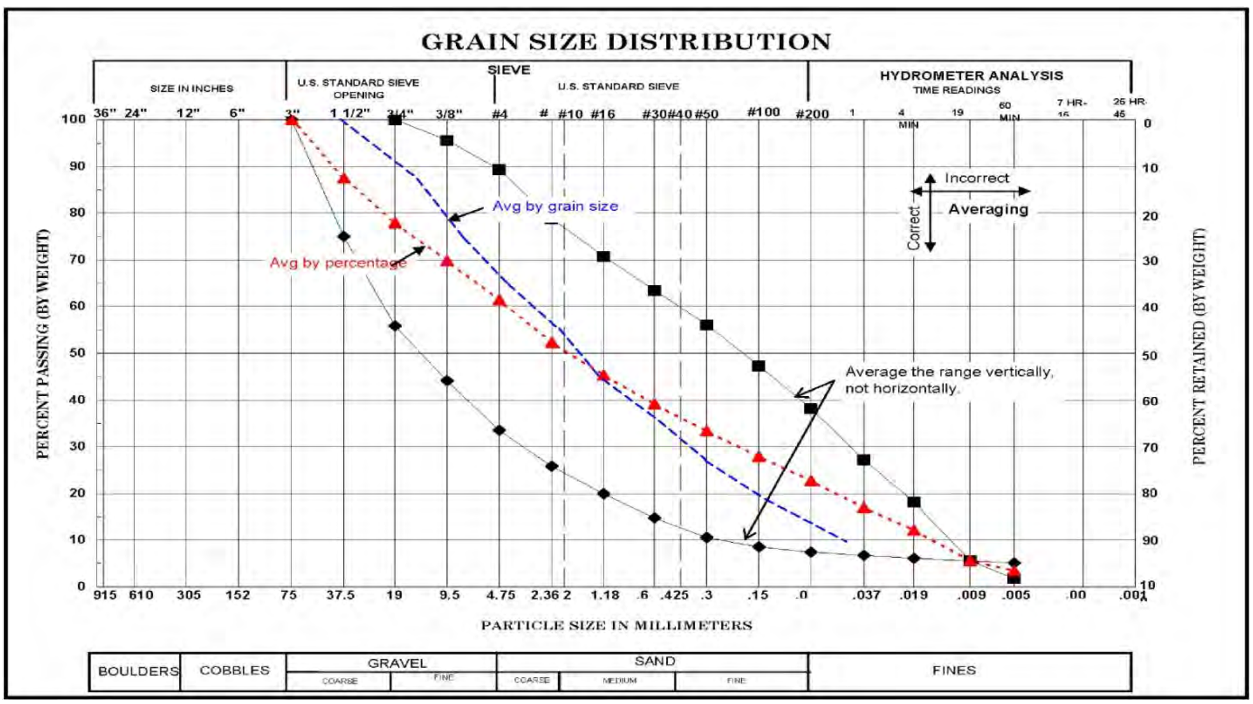

Figure A-17. Example of correct and incorrect method of averaging two grain size curves.

Attachment B –USACE Filter Design Excerpt

From EM 1110-2-2300, dated 30 July 2004:

EM 1110-2-2300
30 JUL 2004

B-9. Example of Graded Filter Design for Drain

Seldom, if ever, is a single graduation curve representative of a given material. A material is generally represented by a gradation band which encompass all the individual gradation curves. Likewise, the required gradation for the filter material is also given as a band. The design of a graded filter which shows the application of the filter criteria where the gradation are represented by bands is illustration is Figure B-2. A typical two-layer filter for protecting an impervious core of a dam illustrated. The impervious core is a fat clay (CH) with a trace of sand which falls in Category 1 soil in Table B-2. The criterion $D_{15} \leq 9 \times d_{85}$ is applied and point "a" is established in Figure B-2. Filter material graded within a band such as that shown for filter material A in Figure B-2 is acceptable based on the stability criteria. The fine limit of the band was arbitrarily drawn and in this example, is intended to represent the gradation of a readily available material. A check is then made to ensure that the 15 percent size of the fine limit of the filter material band (point b) is equal to or greater than 3 to 5 times the 15 percent size of the coarse limit of the drained material band (point c). Filter A has a minimum D_{10} size and a maximum D_{90} size such that, based on Table B-3, segregation during placement can be prevented.

Table B-2. Criteria for Filters

Base Soil Category	Base soil description, and percent finer than No. 200 (0.075mm) sieve[1]	Filter criteria in terms of maximum D_{15} size[2]	Note
1	Fine silts and clays; more than 85% finer.	$D_{15} \leq 9 \times d_{85}$	(1)
2	Sands, silts, clays, and silty and clayey sands; 40 to 85% finer.	$D_{15} \leq 0.7$ mm	//
3	Silty and clayey sands and gravels; 15to 39% finer.	$D_{15} \leq \frac{40-A}{40-15}$ $\{(4 \times d_{85}) - 0.7 \text{ mm}\}$ $+ 0.7$ mm	(2), (3)
4	Sands and gravels; less than 15% finer.	$D_{15} \leq 4$ to $5 \times d_{85}$	(4)

[1] Category designation for soil containing particles larger than 4.75 mm is determined from a gradation curve of the base soil which has been adjusted to 100% passing the No.4 (4.75 mm) sieve.

[2] Filters are to have a maximum particle size of 3 in. (75 mm) and a maximum of %5 passing the No. 200 (0.075 mm) sieve with the plasticity index (PI) of the fines equal to zero. PI is determined on the material passing the No. 40 (0.425 mm) sieve in accordance with EM 1110-2-1906. To ensure sufficient permeability, filters are to have a D_{15} size equal to or greater than $4 \times d_{15}$ but no smaller than 0.1 mm.

NOTES:

(1) When $9 \times d_{85}$ is less than 0.2 mm.
(2) A = percent passing the No. 200 (0.075 mm) sieve after any regrading.
(3) When $4 \times d_{85}$ is less than 0.7 mm, use 0.7 mm.
(4) In category 4, the d_{85} can be based on the total base soil before regrading. In category 4, the $D15 \leq 4 \times d_{85}$ criterion should be used in the case of filters beneath riprap subject to wave action and drains which may be subject to violent surging and/or vibration.

Table B-3. D10 and D90 Limits for Preventing Segregation

If Minimum D_{10}, mm	Then Maximum D_{90}, mm
<0.5	20
0.5 – 1.0	25
1.0 – 2.0	30
2.0 – 5.0	40
5.0 – 10	50
10 – 50	60

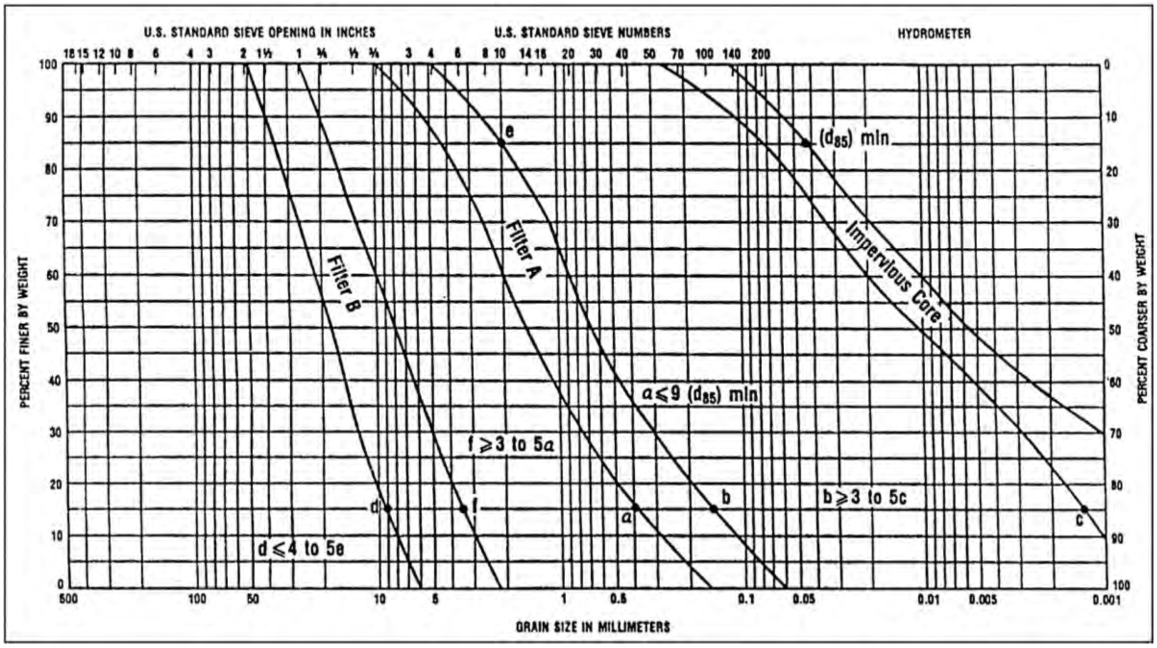

Figure B-2. Illustration of the design of a graded filter.

Attachment C – Example Borrow Area Grain Size Analysis

Background

In this example, filter and drainage materials are required for construction of a toe drain. Two distinct foundation units (base soils) are present at the site and a filter designed for each unit. A single drainage material is used to encapsulate the perforated drainage pipe.

The borrow area is onsite and consists of fines, sands, gravels, cobbles, and boulders. Since the materials are not evenly distributed within the borrow (i.e., the volume of sand does not equal the volume of gravel), a sieve-by-sieve analysis is required to assure that all material sizes are present in sufficient quantities. The following steps outline the procedure for the sieve-by-sieve analysis.

Step 1 Determine the volume of material needed for construction and the volume of raw material available from the borrow area. For this example, the volume of material required is taken from the quantities estimate based on the design drawings. The quantity of the borrow area was estimated as part of the borrow area exploration program. The available and required volumes are summarized in Table C-1. In order to convert from volume to weight, it was assumed that all materials have a unit weight of 125 lb/cu ft.

Table C-1. Volumes of material required and available.

–	Material			–
Unit	Filter 1	Filter 2	Drain	Borrow Area
Volume – cu yd	96,180	29,400	7,560	470,000
Volume – cu ft	2,596,860	793,800	204,120	12,657,242
Unit Weight – lb/cu ft	125	125	125	125
Weight – lb	342,607,500	99,225,000	25,515,000	1,586,250,000
Weight – tons	162,304	49,613	12,758	793,125

Step 2 Determine the gradation of the borrow area using the average of all gradation tests. For the proposed borrow, this average gradation is shown in Table C-2 under Column A. This table is used to separate the oversize (> 3-in.) material out of the borrow pit in a manner similar to what would be done in the processing plant. Since no material greater than 3 in. is required to produce the filter and drain materials, it is scalped off before entering the plant. The procedure used to remove this oversize material from the bulk gradation is the same as the re-grading procedure described in section 5.1.3. As shown in Column A, 74% of the material is finer than the 3 in. (26% are cobbles or larger). Therefore, the weight of material less than 3 in. is 793,125 × 0.74 = 586,912.

Table C-2. Weight of material available by sieve from borrow area.

			Column A	Column B	Column C	Column D	Column E	Column F
Size		Diameter (mm)	Cumulative Percent Passing	Size Range per Sieve	Total Sample Percent Retained	Cumulative Minus 3 in. Percent Passing	Minus 3 in. Avg. Percent Retained (Borrow Area)	Available Weight of Borrow Area (ton)
U.S. Standard Sieve Opening in in.	72 in.	1800	100.0	1 ft - 6 ft’	10.5			N/A
	12 in.	300	89.5	3 in. – 1 ft	15.5		0.0	N/A
	3 in.	75	74.0	1-1/2 in. – 3 in.	4.8	100.0	6.5	38,149
	1-1/2 in.	37.5	69.2	3/4 in. – 1-1/2 in.	6.3	93.5	8.5	49,888
	3/4 in.	19	62.9	3/8 in. - 3/4 in.	6.7	85.0	9.0	52,822
	3/8 in.	9.5	56.2	No. 4 - 3/8 in.	6.8	76.0	9.2	53,996
U.S. Standard Sieve Numbers	No. 4	4.75	49.4	No. 8 - No. 4	4.9	66.8	6.6	38,736
	No. 8	2.36	44.5	No. 16 - No. 8	5.8	60.2	7.9	46,366
	No. 16	1.18	38.7	No. 30 - No. 16	6.9	52.3	9.3	54,583
	No. 30	0.6	31.8	No. 50 - No. 30	8.4	43.0	11.3	66,321
	No. 50	0.3	23.5	No. 100 - No. 50	8.3	31.7	11.2	65,734
	No. 100	0.15	15.2	No. 200 - No. 100	4.9	20.5	6.6	38,736
	No. 200	0.075	10.3	PAN - No. 200	10.3	13.9	13.9	81,581
						TOTAL	100.0	586,912

Column A Cumulative % passing per sieve based on gradation tests of multiple samples taken of the borrow area material.
Column B Size range of material retained on any particular sieve.
Column C Percent of the entire sample retained per sieve.
Column D Cumulative % of the re-graded (minus 3 in.) retained per sieve.
Column E Percent of re-graded sample retained per sieve.
Column F Weight of material retained per sieve. Column E × (0.74 × 793,125).

Step 3 Determine the amount of material required, per sieve, for filter 1. The specified gradation for filter 1 is presented in Table C-3 under Column A. Table C-1 showed that 162,304 tons of filter 1 were required for the work.

Table C-3. Weight of material required by sieve for filter 1.

//	//	//	Column A	Column B	Column C	Column D
-	Size	Diameter (mm)	Cumulative Percent Passing	Size Range per Sieve	Percent Retained Filter 1	Required Height of Filter 1 (tons)
U.S. Standard Sieve Opening in inches	3 in.	75	-	1-1/2 in – 3in.	-	-
	1 - ½in.	37.5	-	¾ in – 1 ½ in.	0.0	0
	¾ in.	19	100.0	3/8 in – ¾ in.	2.4	3,895
	3/8 in	9.5	97.6	No. 4 – 3/8 in.	12.1	19,639
U.S. Standard Sieve Numbers	No. 4	4.75	85.5	No. 8 – No. 4	21.3	34,571
	No. 8	2.36	64.2	No. 16 – No. 8	21.4	34,733
	No. 16	1.18	42.8	No. 30 – No. 16	21.4	34,246
	No. 30	0.6	21.7	No. 50 – No. 30	16.3	26,456
	No. 50	0.3	5.4	No. 100 – No. 50	5.4	8,764
	No. 100	0.15	0.0	No. 200 – No. 100	-	-
	No. 200	0.075	-	PAN - No. 200	-	-
-	-	-	-	TOTAL	100.0	162,304

Column A Cumulative% passing per sieve based on the average gradation from the specification.
Column B Size range of material retained on any particular sieve.
Column C Percent of sample retained per sieve.
Column D Weight of material retained per sieve.

Step 4 Determine the amount of material required, per sieve, for filter 2. The specified gradation for filter 2 is presented in Table C-4 under Column A. Table C-1 showed that 49,613 tons of filter 2 were required for the work.

Table C-4.—Weight of material required by sieve for filter 2.

			Column A	Column B	Column C	Column D
Size		Diameter (mm)	Cumulative Percent Passing (Zone 2)	Size Range per Sieve	Percent Retained (Filter 2)	Required Weight of Filter 2 (tons)
U.S. Standard Sieve Opening in in.	3"	75		1-1/2 in. - 3 in.		
	1-1/2 in.	37.5		3/4 in. - 1-1/2 in.	0.0	0
	3/4 in.	19	100.0	3/8 in. - 3/4 in.	3.7	1,836
	3/8 in.	9.5	96.3	No. 4 - 3/8 in.	15.0	7,442
U.S. Standard Sieve Numbers	No. 4	4.75	81.3	No. 8 - No. 4	30.7	15,231
	No. 8	2.36	50.6	No. 16 - No. 8	29.6	14,685
	No. 16	1.18	21.0	No. 30 - No. 16	17.0	8,434
	No. 30	0.6	4.0	No. 50 - No. 30	4.0	1,985
	No. 50	0.3	0.0	No. 100 - No. 50		
	No. 100	0.15		No. 200 - No. 100		
	No. 200	0.075		PAN - No. 200		
				TOTAL	100.0	49,613

Column A Cumulative percent passing per sieve based on the average gradation from the specification.
Column B Size range of material retained on any particular sieve.
Column C Percent of sample retained per sieve.
Column D Weight of material retained per sieve.

Step 5 Determine the amount of material required, per sieve, for the drain material. The specified gradation for the drain material is presented in Table C-5 under Column A. Table C-1 showed that 12,758 tons of drain material were required for the work.

Table C-5. Weight of material required by sieve for the drain.

			Column A	Column B	Column C	Column D
Size		Diameter (mm)	Cumulative Percent Passing	Size Range per Sieve	Percent Retained (Drain)	Required Weight of Drain (tons)
U.S. Standard Sieve Opening in in.	3 in.	75	100.0	1-1/2 in. - 3 in.	7.5	957
	1-1/2 in.	37.5	92.5	3/4 in. – 1-1/2 in.	32.5	4,146
	3/4 in.	19	60.0	3/8 in. - 3/4 in.	33.9	4,325
	3/8 in.	9.5	26.1	No. 4 - 3/8 in.	26.1	3,330
U.S. Standard sieve Numbers	No. 4	4.75	0.0	No. 8 - No. 4		
	No. 8	2.36		No. 16 - No. 8		
	No. 16	1.18		No. 30 - No. 16		
	No. 30	0.6		No. 50 - No. 30		
	No. 50	0.3		No. 100 - No. 50		
	No. 100	0.15		No. 200 - No. 100		
	No. 200	0.075		PAN - No. 200		
				TOTAL	100.0	12,758

Column A Cumulative% passing per sieve based on the average gradation from the specification.
Column B Size range of material retained on any particular sieve.
Column C Percent of sample retained per sieve.
Column D Weight of material retained per sieve.

Step 6 Deduct the weights of filter 1, filter 2, and the drain materials from the borrow area as shown in Table C-6.

Table C-6. Balance between supply and demand for the borrow area.

		Column A	Column B	Column C	Column D	Column E	Column F	Column G
U.S. Standard Sieve Opening in in.	Size	Required Amount of Filter 1 (tons)	Required Amount of Filter 2 (tons)	Required Amount of Drain (tons)	Total Required Weight (tons)	Available Amount of Borrow (tons)	Waste (tons)	E / D
	1-1/2 in. - 3 in.			957	957	38,149	37,094	39.9
	3/4 in. -1-1/2 in.	0	0	4,146	4,146	49,888	45,612	12.0
	3/8 in. - 3/4 in.	3,895	1,836	4,325	10,056	52,822	42,630	5.2
	No. 4 - 3/8 in.	19,639	7,442	3,330	30,411	53,996	23,446	1.8
U.S. Standard Sieve Numbers	No. 8 - No. 4	34,571	15,231		49,802	38,736	-11,166	0.81
	No. 16 - No. 8	34,733	14,685		49,419	46,366	-3,172	0.91
	No. 30 - No. 16	34,246	8,434		42,680	54,583	11,762	1.3
	No. 50 - No. 30	26,456	1,985		28,440	66,321	37,710	2.3
	No. 100 - No. 50	8,764			8,764	65,734	56,800	7.5
	No. 200 - No. 100				0	38,736	38,636	N/A
	PAN - No. 200				0	81,581	81,370	N/A
	TOTAL	162,304	49,613	12,758	224,675	585,912	360,723	

Column A From Column D of Table C-3.
Column B From Column D of Table C-4.
Column C From Column D of Table C-5.
Column D A + B + C.
Column E E - D.
Column E E / D.

These data indicate that while only 264,000 cubic yards of processed material is required for the work and 586,000 are available in the borrow area, the borrow area is deficient in the medium sand sizes (No. 16 to No. 4 sieves). Not being aware of this fact could lead to a contractor making a claim during execution of the work since insufficient material is available to produce the filters. Also notice that this processing operation will result in about 50% byproduct (waste) of 3/4- to 3-in. material. Possible solutions to this situation include:

- Add a crushing operation to make medium sand from 3/4-in. gravel
- Maximize the size of the drain envelope around the pipe
- Utilize the 1-1/2- to 3-in. material as slope protection and riprap bedding in conjunction with the 24-percent oversize (> 3-in.) material.

Table C-6 illustrates that 81,370 tons, or about 50,000 cu yd, of fines (washer waste) are produced from this operation. This material will have to be re-handled and disposed.

Attachment D – Alternative Method for Limiting Gap-Graded Filter Gradation

The filter design procedure presented in Chapter 5 describes a method that minimizes the chance of a gap-graded filter from being produced. That method is to restrict the gradation band width based on the D_{60} properties, which can be thought of as a horizontal control on the gradation limits. In a similar manner, the gradation limits can be controlled vertically. This is done by not permitting the difference of the lower limit of percent passing and the upper limit percent passing to be greater than 35 percentage points. This is shown graphically in Figure D-1. In this figure, a vertically "sliding bar" is used to constrain the gradation. The bar is noted by points L and M and is shown at the maximum 35-point length. The bar can be moved around the gradation plot, but point L cannot move to the left of a line drawn between points A and K and cannot move any further to the right than point B. As described in Chapter 5, the L-M bar can be moved around to a location compliant with the intended use of the filter. For finer grained filters intended to focus on particle retention, the bar would be positioned to the right. Coarser filters focusing on permeability would be positioned to the left.

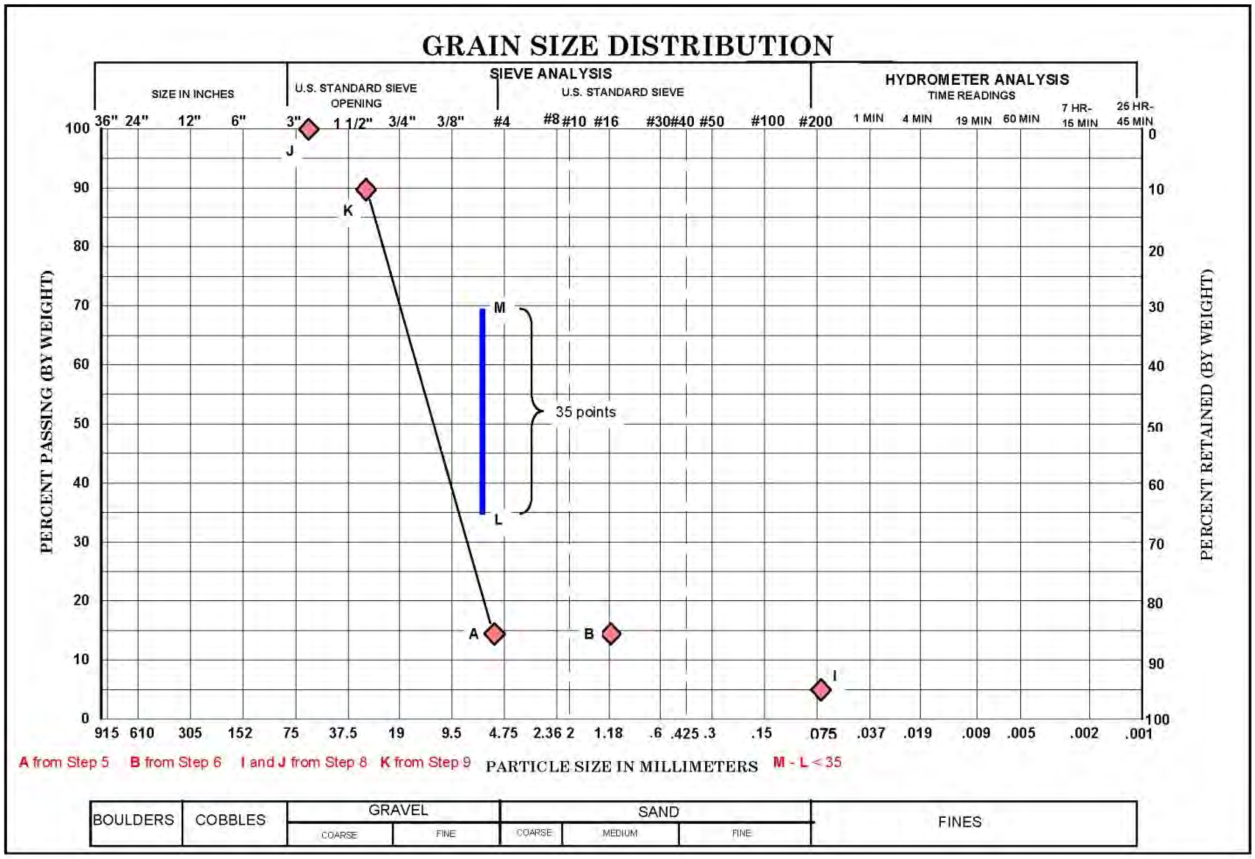

Figure D-1. Example gradation limits to address gap-graded materials.

Attachment E – Case Histories

Tallaseehatchie Creek Site 1

Location: Clay County, Alabama

Summary

Overtopping of embankment during construction washed out chimney filter and most of the downstream embankment rockfill zone.

Tallaseehatchie Creek Site 1 is a multipurpose (flood control and water supply) reservoir constructed in Clay County, Alabama, by the Natural Resources Conservation Service in 1978-79.

During the winter shutdown period, the embankment was only partially completed. To provide temporary protection, a small dike was built across the embankment and a small bypass spillway was left in the right abutment. The dike and bypass spillway were sized to bypass a 10-yr, 24-hr storm, assuming flow through the 50-ft-wide bypass spillway was 5.2 ft deep.

Figure E-1 shows the cross-section of the embankment centerline at winter shutdown time. At the end of the dike, in the right abutment, a 50-ft-wide notch about 5.2 ft deep was left.

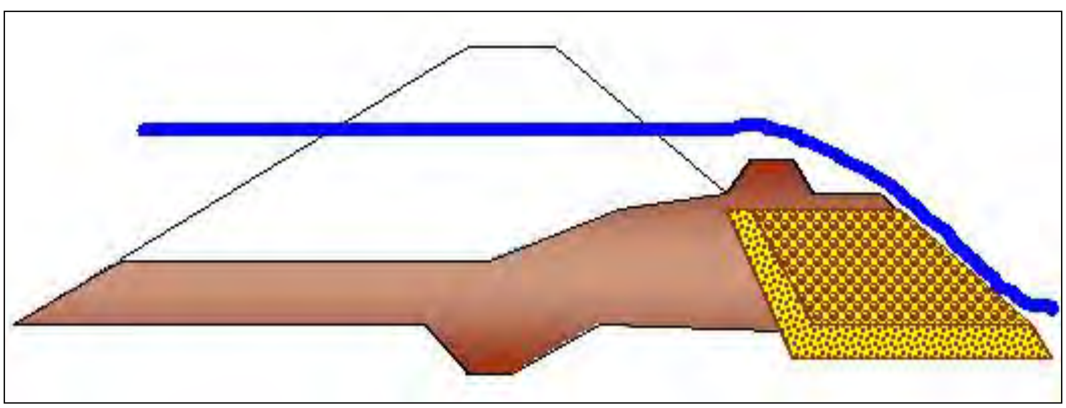

Figure E-1. Cross-section of embankment at time of overtopping.

A large rainfall event occurred that caused the dike to overtop during the shutdown period. The storm was estimated to be an 18-yr, 24-hr storm, and water was estimated to have flowed over the dike to a depth of about 2.5 ft (this would have meant water flowing to a depth of about 7.7 ft in the 5.2 ft deep spillway). Erosion of the dike occurred in about a 50-ft-wide section near the left abutment and about 100 ft in the right abutment where the auxiliary spillway notch was left.

The overtopping flow eroded almost the entire downstream rockfill zone and most of the chimney filter zone. Pictures of the damage are shown in Figures E-2 through E-6.

Lessons learned

Filter/drainage zones are composed of relatively fine, cohesionless sand gradations that can be expected to be highly erodible. Protecting these zones from overtopping by covering them with an adequate depth of erosion-resistant material is important during periods of winter shutdown. Overtopping eroded over 60,000 cu yd of material.

Figure E-2. View toward left abutment. This 50-ft-wide section of the dike overtopped and eroded a portion of the dike.

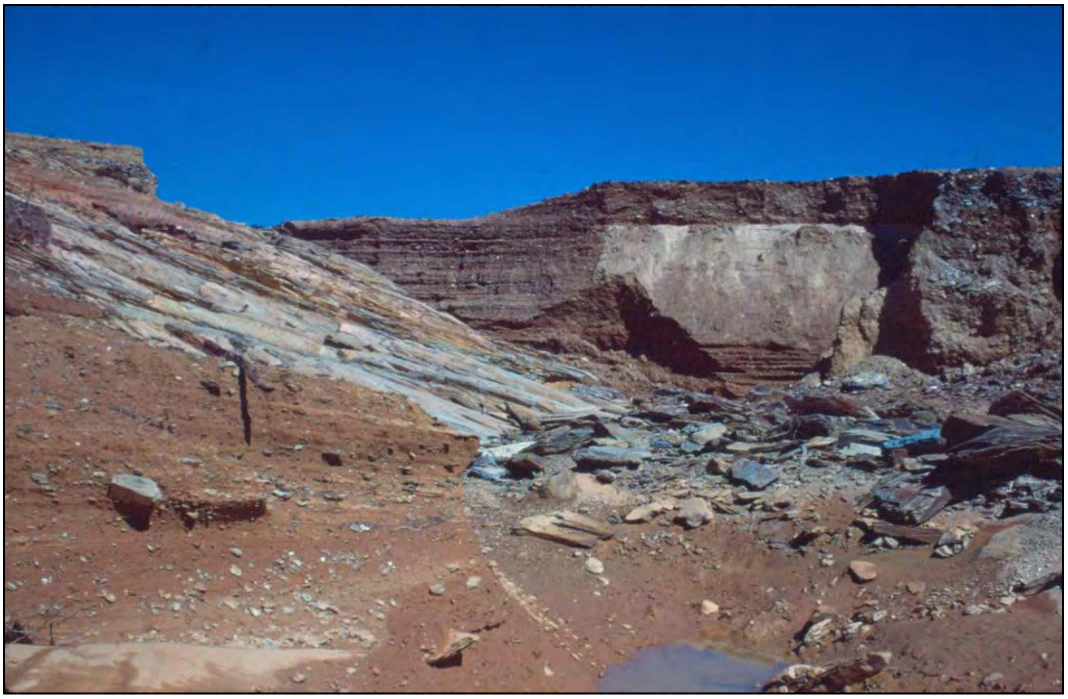

Figure E-3. Breach in left abutment viewed in upstream direction. Right abutment is to left foreground. Overtopping flow washed the embankment and overburden down to clean rock.

Figure E-4. View toward right abutment showing remnant of chimney filter/drain in cross-sectional view. Chimney filter/drain was 3 ft wide.

Figure E-5. Closeup of right abutment viewed upstream. Bedrock was cleaned by erosion from overtopping flow. Embankment is in top right of photograph.

Figure E-6. Breached portion of dike in left abutment viewed upstream. The lighter color is remnant of a chimney filter.

Narrow toe drain

Location: Idaho

Summary

As part of a safety of dams modification to a 100-yr-old dam, a large toe drain system was added to address deficiencies associated with pervasive seepage through the foundation. Due to the size of the repair, and in the interest of keeping costs low, a modest cross-section was used, as shown in Figure E-7.

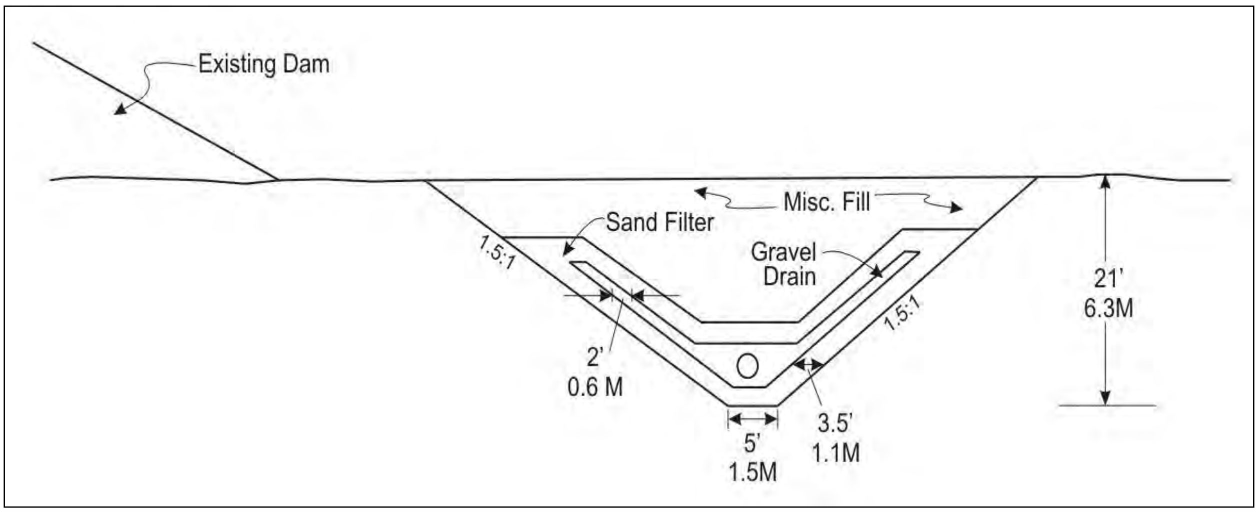

Figure E-7. Toe drain configuration at the end of construction.

Upon first filling, silt and sand were detected in the sedimentation traps that were included in the inspection wells added during the modification. The rate at which material was collecting in the sedimentation traps, along with the cloudy color of the collected flow, indicated the new drainage system had failed in some way. A forensic investigation was undertaken in the form of removing portions of the new drain system. That investigation led to the following understanding of what had happened.

As shown on Figure E-8, the filter layer against the foundation was found to be less than the specified width and, in some places, was completely missing. It has been speculated that when the trackhoe rotated, the back of the cab would run into previously placed filter material. It is also possible that equipment travel along the trench, as well as entering and exiting the trench, could have led to removal of the filter layer against the foundation.

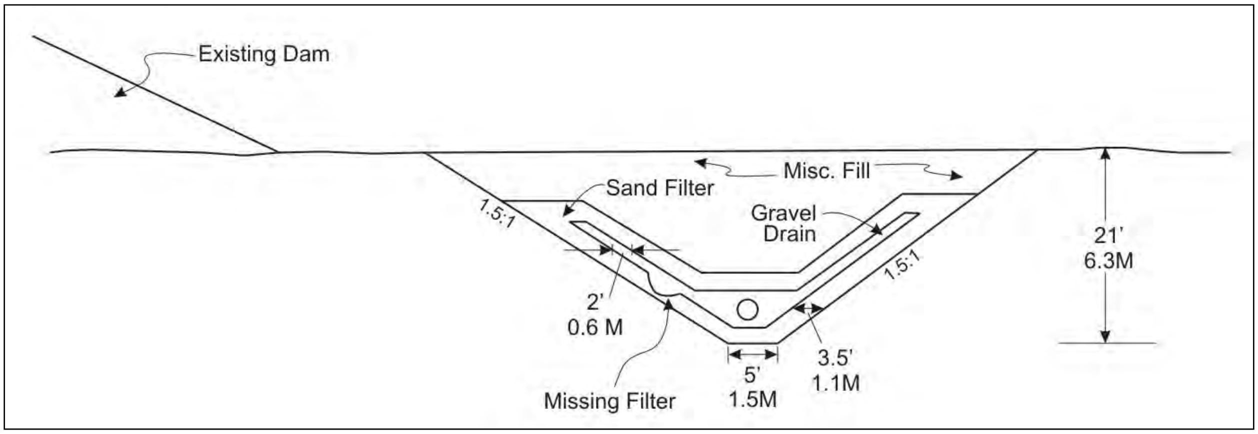

Figure E-8. Area of possible filter damage.

Construction was performed in the winter months while the reservoir was low, and the limited hours of daylight resulted in some construction at night. While continuous onsite construction inspection was performed by the owner, the damage was not detected by staff.

Since the gravel drain was in direct contact with the foundation in some places and the foundation contains silts, sands, and gravels, filter compatibility was not met. Therefore, silt and sand were able to erode (pipe) into the gravel drain as shown on Figure E-9.

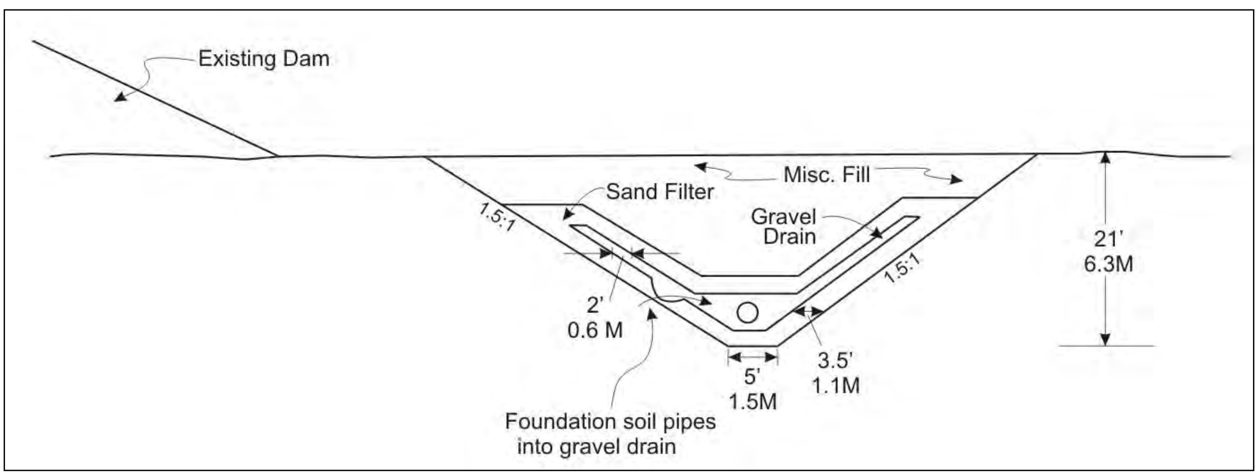

Figure E-9. Piping of foundation soil into gravel drain.

Material transport continued through the gravel drain and through the perforations in the drainage pipe. The flow in the pipe then carried the material to the sediment trap where it was identified during re-fill monitoring. Material transfer into the pipe is illustrated on Figure E-10.

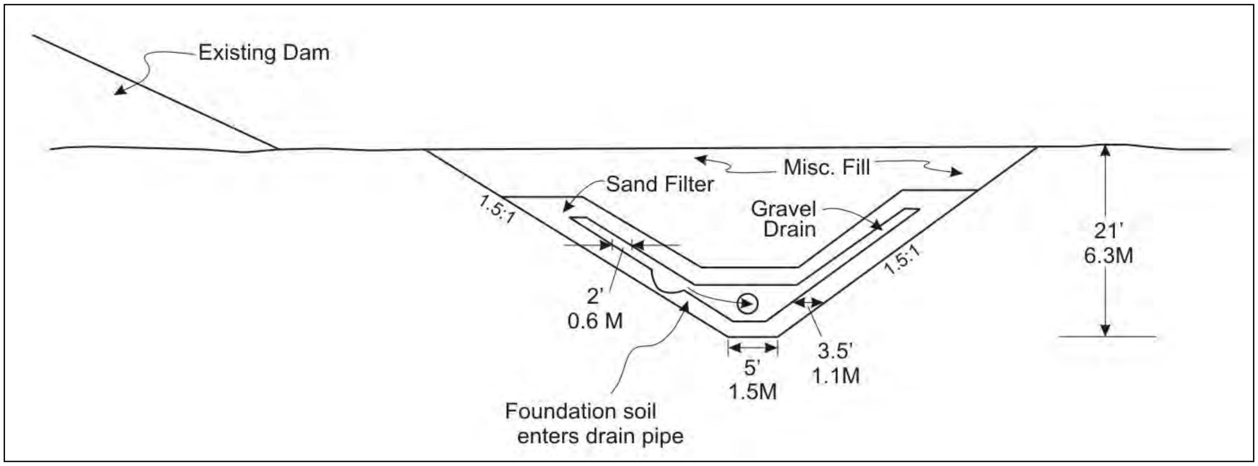

Figure E-10. Foundation soil passes through gravel drain and enters drain pipe.

Figure E-11 illustrates the problems with the toe drain design. The narrow bottom width made it impractical for commonly available construction equipment to work in the bottom of the trench. The 21-ft depth made it impractical to work from the top to place initial lifts in the bottom of the trench. Trenches should always be sized so equipment can work from inside the trench and not from the top. Relatively steep sideslopes were used and, while material could be placed and compacted on this slope, traffic up and down the slope would damage the surface. Lastly, narrow filter and gravel drain zones were used that were difficult to place and prone to damage.

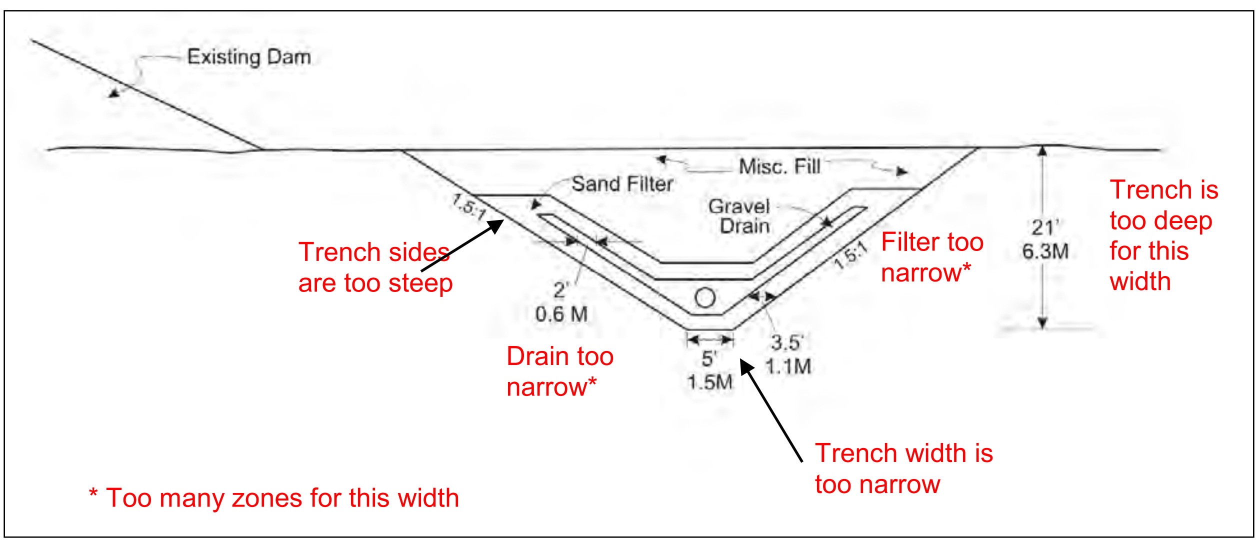

Figure E-11. Poor toe drain design elements.

Lessons learned

Construction inspection may not necessarily overcome poor design elements. Minimum toe drain geometry, as described in this manual, should be used so a contractor can construct it with a reasonable amount of effort.

Sinkhole at Chilhowee Dam

Location: Knoxville, Tennessee

Summary

The Chilhowee Hydroelectric Project located on the Little Tennessee River near Knoxville, Tennessee, is owned by Alcoa Power Generating, Inc. (APGI), and is licensed by the Federal Energy Regulatory Commission. PB Power was the Owner's Engineer during this sinkhole evaluation. Constructed in 1957, approximately 40% of the total dam length is comprised of the north and south rockfill embankments, each having a sloping clay core bounded upstream and downstream by filter layers and rockfill shells, as shown in Figure E-12.

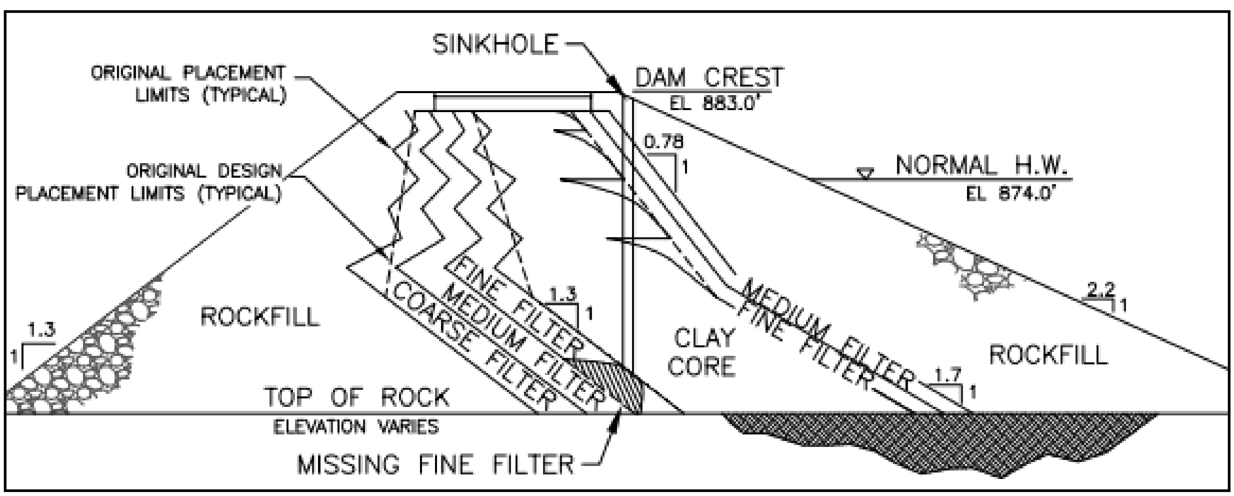

Figure E-12. Cross section of the original embankment construction showing sinkhole and missing fine filter. Note the "Herringbone" pattern of the downstream filters; original construction placement techniques caused the as-built filter locations (solid line) to deviate from the design placement limits (dashed line).

A 6-in.-diam sinkhole was discovered on the southern embankment near the south abutment in 2000. More than 4 ft of settlement was measured at the sinkhole over the next eight years. A range of subsurface geotechnical investigations was used to investigate the cause and extent of the sinkhole. Although this effort identified several anomalies within the embankment, the investigation was not able to determine the sinkhole's cause and extent.

It was then decided to excavate and rebuild the affected area of the embankment after the subsurface investigations showed there were deficient areas of the core in the vicinity of the sinkhole, an effort which would involve lowering the reservoir more than 25 ft below the normal headwater level. The rebuild began in September 2008 with a careful investigatory excavation of the sinkhole to follow the location and identify the causes of the sinkhole.

This careful excavation found that the sinkhole terminated in a small area next to the rock abutment where the clay core was in direct contact with the medium filter. During original construction, the design called for the embankment filters to be graded so that adjacent filters and clay core would be filter compatible with each other to prevent piping. The missing fine filter allowed clay to migrate through the medium filter over time, as shown in Figure E-13, leading to the development of the sinkhole.

Figure E-13. Comparison between normal gray medium filter (left) and medium filter contaminated with red-brown clay from near the missing fine filter.

A vertical crack (likely caused by an overhanging face in the rock abutment) and several air voids in the clay core were also identified during the controlled excavation. The sinkhole and approximate 1-ft-wide clay crack terminated at the missing fine filter.

Gradations for the materials used during the embankment rebuild were chosen to match the specifications of the original construction documents and maintain filter compatibility between all adjacent new and existing materials. Filter compatibility was determined based on the U.S. Army Corps of Engineers' Engineering Manual EM1110-2-2300 Appendix B, Filter Design Criteria (30 July 2004). Two new filters were added to the rebuild design, which had not been included in the original construction. A coarse filter was added upstream of the upstream medium filter to maintain filter compatibility between the new upstream rockfill and medium filter material. A sand filter was added between the clay core and downstream fine filter for added redundancy, as shown in Figure E-14.

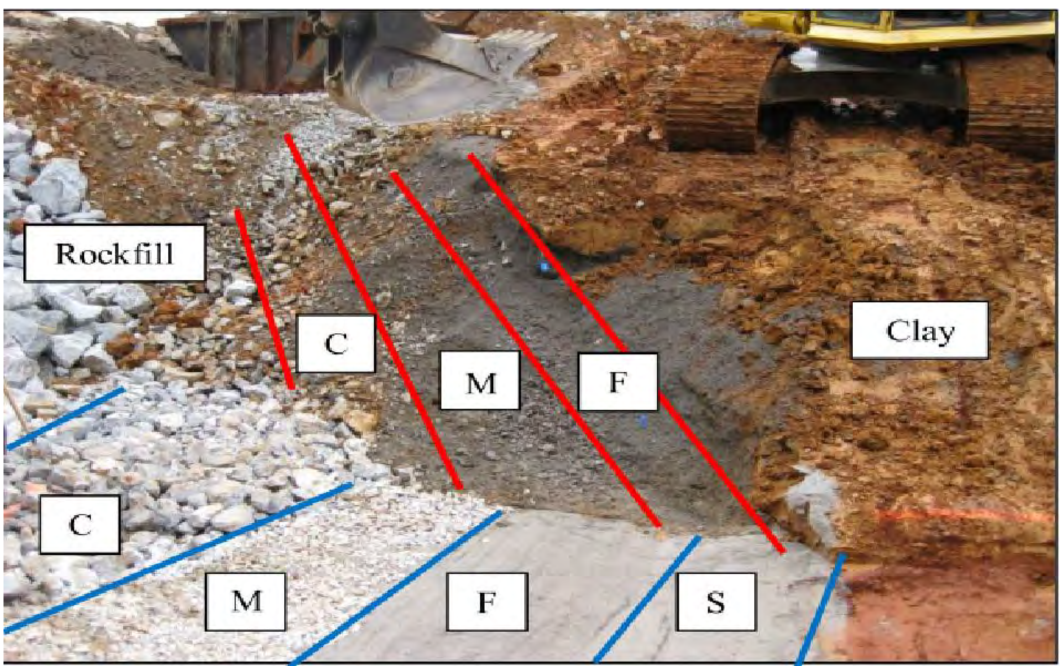

Figure E-14. Filter survey and placement considerations at existing embankment interface. (Filters: S = Sand, F = Fine, M=Medium, C=Coarse).

An onsite soil testing laboratory was used to ensure that clay and filters met the specified soil properties and material gradations. Field Engineer oversight ensured that filter compatibility and required filter widths were maintained. However, due to the addition of the new filter layers, the disparity between the design placement limits of the existing embankment, and the actual as-built position of the filters, a custom transition was developed over the last 10 to 20 ft of the rebuild to ensure the newly placed embankment material met compatibility requirements with the existing embankment material. The main component of placing each lift of filter material was maintaining filter compatibility between the newest 1 ft (precompaction) vertical lift of embankment material and (1) existing filter material at the limits of the excavation and (2) previous lift of filter material below the newest lift while seeking to place materials as close to their intended design limits as possible.

Lessons learned

Localized piping of the clay core was caused by a zone of missing fine filter material adjacent to, and downstream of, the impervious clay core. A crack through the clay core, caused by the overhanging rock abutment, allowed concentrated seepage through the embankment at the missing filter. Clay contamination was noticed in the thin zone of fine filter as it pinched out next to the missing fine filter zone. Medium and coarse filter materials, as well as the rockfill, had noticeable clay contamination near the missing fine filter.

Geotechnical borings conducted near the sinkhole narrowly missed intersecting the clay crack, which was infilled with embankment material. During the careful excavation, grout columns from the boring program were observed within a few ft of the clay crack. Geophysics identified low velocity zones within the clay core which, upon excavation, coincided with areas of noticeably softer clay, voids in the clay, the sinkhole, and the clay crack.

Due to the difficulty of matching rebuild placement limits to original construction, it was helpful to incorporate a transition zone to allow a majority of the rebuild to follow the filter construction design limits before changing direction to tie into the existing embankment.

Full-time construction quality control by a knowledgeable and experienced Field Engineer is key to avoiding a small zone of missing or unacceptable filter material as well as ensuring proper clay placement. This is particularly crucial in the first filter downstream of the impervious core material, which protects the core from piping.

Attachment F – Laboratory Filter Test Procedures

Reclamation filter testing

cover page from:
U.S. Department of the Interior
Bureau of Reclamation
USBR 5630-89

UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
Appendix 2.4A - Filter Testing Procedure US Bureau Reclamation

USBR 5630-89

PROCEDURE FOR

PERFORMING FILTER TESTING

INTRODUCTION

This procedure is under the jurisdiction of the Geotechnical Services Branch, code D-3760, Research and Laboratory Services Division, Denver Office, Denver, Colorado. The procedure is issued under the fixed designation USBR 5630. The number immediately following the designation indicates the year of acceptance or the year of last revision.

1. Scope

1.1 This designation outlines the laboratory procedure for evaluating suitability of a soil material or geotextile to perform as a filter.

1.2 The procedure is applicable for soil intended to perform as filters for pipe drains, canal underdrains and linings, and filter zones in the foundation and internal zonings of earth embankment dams.

1.3 The procedure is used to determine if a soil or geotextile can prevent unrestrained erosion of an adjacent material that has a finer particle size distribution, while maintaining adequate hydraulic conductivity to prevent excess hydrostatic pressure.

2. Auxiliary Tests

2.1 Test samples must be prepared in accordance with USBR 5205 prior to performing this procedure. The moisture content, particle size distribution, liquid limit, plastic limit and plasticity index, specific gravity, moisture-unit weight relationship, and/or minimum and maximum index unit weights of the soils must be determined in accordance with USBR 5300, 5325 and/or 5330, 5320, 5360, 5500 and/or 5525, and 5530, respectively. These tests must be performed to prepare test specimens and to select placement conditions for the filter test. Permeability of soils should be determined in accordance with USBR 5600, 5605, and/or 5610. Durability of soil filter materials should be evaluated prior to performing the filter test to identify friable materials that could undergo significant particle-size breakdown due to construction processes and thereby diminish their effectiveness as a filter. The Los Angeles abrasion test [1][1] and/or the sand attrition test [2] can provide information about the breakdown of a soil filter material. The soundness or ability of sands and gravels to resist weathering can be determined by sodium sulphate testing [3].

3. Applicable Documents

3.1 *USBR Procedures*:

USBR 1000 Standards for Linear Measurement Devices
USBR 1010 Calibrating Unit Weight Measures
USBR 1012 Calibrating Balances or Scales
USBR 1020 Calibrating Ovens
USBR 1025 Checking Sieves
USBR 3000 Using Significant Digits in Calculating and Reporting Laboratory Data
USBR 3900 Standard Definitions of Terms and Symbols Relating to Soil Mechanics
USBR 5000 Determining Unified Soil Classification (Laboratory Method)
USBR 5005 Determining Unified Soil Classification (Visual Method)
USBR 5205 Preparing Soil Samples by Splitting or Quartering
USBR 5210 Preparing Compacted Soil Specimens for Laboratory Use
USBR 5300 Determining Moisture Content of Soil and Rock by the Oven Method
USBR 5320 Determining Specific Gravity of Soils
USBR 5325 Performing Gradation Analysis of Gravel Size Fraction of Soils
USBR 5330 Performing Gradation Analysis of Fines and Sand Size Fraction of Soils, Including Hydrometer Analysis
USBR 5350 Determining the Liquid Limit of Soils by the One-Point Method
USBR 5360 Determining the Plastic Limit and Plasticity Index of Soils
USBR 5500 Performing Laboratory Compaction of Soils — 5.5-lbm Rammer and 18-in Drop
USBR 5515 Performing Laboratory Compaction of Soils Containing Gravel
USBR 5525 Determining the Minimum Index Unit Weight of Cohesionless Soils
USBR 5530 Determining the Maximum Index Unit Weight of Cohesionless Soils
USBR 5600 Determining Permeability and Settlement of Soils [8-in (203-mm) Diameter Cylinder]
USBR 5605 Determining Permeability and Settlement of Soils Containing Gravel
USBR 5610 Determining Permeability of Soils by the Back Pressure Test Method
USBR 5890 Preparing Laboratory Bentonitic Slurry Sample
USBR 5891 Determining Coarse-Grained Portion of a Slurry by the Sand Content Test Method
USBR 5894 Determining Slurry Viscosity by the Marsh Funnel Test Method

[1] Number in brackets refers to the reference.

Kohr and Woo no erosion filter test

Kohr and Woo (1989) presented a no erosion filter test that was slightly different than the one devised by Sherard, et al. (1984a). Their test apparatus had different dimensions because the filter being studied was a crushed rock filter that was accommodated better in a slightly larger device. The summary of the test procedure used by Kohr and Woo is as follows:

1. Saturate and thoroughly mix 5,000 grams of the proposed filter sand. Compact 80 millimeters (mm) (about 3 in.) of sand in two equal layers into the test cylinder. If the filter has particles larger than the No. 4 sieve, place sand around the perimeter of the sample for at least the upper 2 in. The test cylinder has an internal diameter of 170 mm (6.7 in.) and is 290 mm (11.4 in.) long, illustrated in Figure F-1.

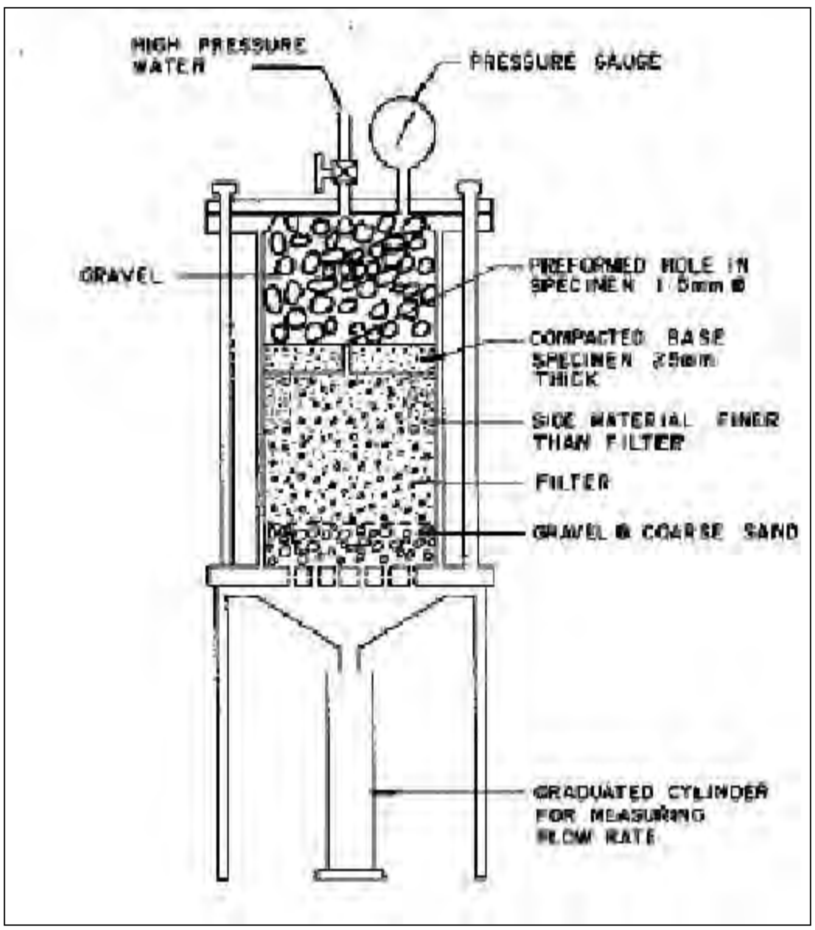

Figure F-1. Filter test details (schematic).

2. Vibrate the sample on a vibrating table like that used for relative density testing for 1 min with an 8-kg (17.6-lb) weight surcharge.
3. With a metal pin that is 1.5 mm in diameter held in position at the center of the cylinder, compact about 1,100 grams (2.4 lb) of the base soil that is being tested against the filter for compatibility to a layer that is about 25 mm (1 in.) thick around the metal pin. Use a tamper and energy to approximate 95% of standard Proctor (ASTM D698) dry density.
4. Withdraw the pin from the compacted base soil specimen to form the hole for the initial leakage channel. Fill the space above the base soil layer with gravel and then assemble the top plate on the test cylinder. Fill the cylinder with water, displacing air through a hole in the top plate.
5. After attaching the test apparatus to a water supply with water pressure of about 4 kg/cm^2 (57 $lb/in.^2$), open the valve supplying water pressure abruptly. Water will flow through the preformed hole into the test filter.
6. Collect water emerging from the bottom of the test cylinder and observe the flow rate for at least 10 min, recording the quantity of water discharged and the qualitative turbidity of the water.
7. Turn off the water supply and dismantle the apparatus. Examine the walls of the preformed pinhole and the filter face for signs of erosion.
8. Judge a test as successful when there is little visible sign of erosion of the hole in the base specimen and the turbidity of the collected flow is low. Tests with successful filters should show a steady flow rate of less than 200 ml/min.

Soil Conservation Service no erosion filter test

1. The filter was placed in three layers in a moist condition to minimize segregation.
2. Compacted to a moderately dense state by heavy vibration (on a vibrating table) with a 20-lb cylindrical steel weight surcharge, having exterior diameter about 1/8 in. smaller than the inside diameter of the 4-in.-diam plastic cylinder.
3. The base was compacted by tamping to a moderately dense condition at a water content judged visually to be satisfactory for good compaction (near standard Proctor optimum water content).
4. The tests were all made with the cylinder in the vertical position with downward water flow as shown in Figure F-1.
5. The 1.0 millimeter (mm) diam hole was made through the base specimen by punching with a hypodermic needle, using the same procedure as for a pinhole test.

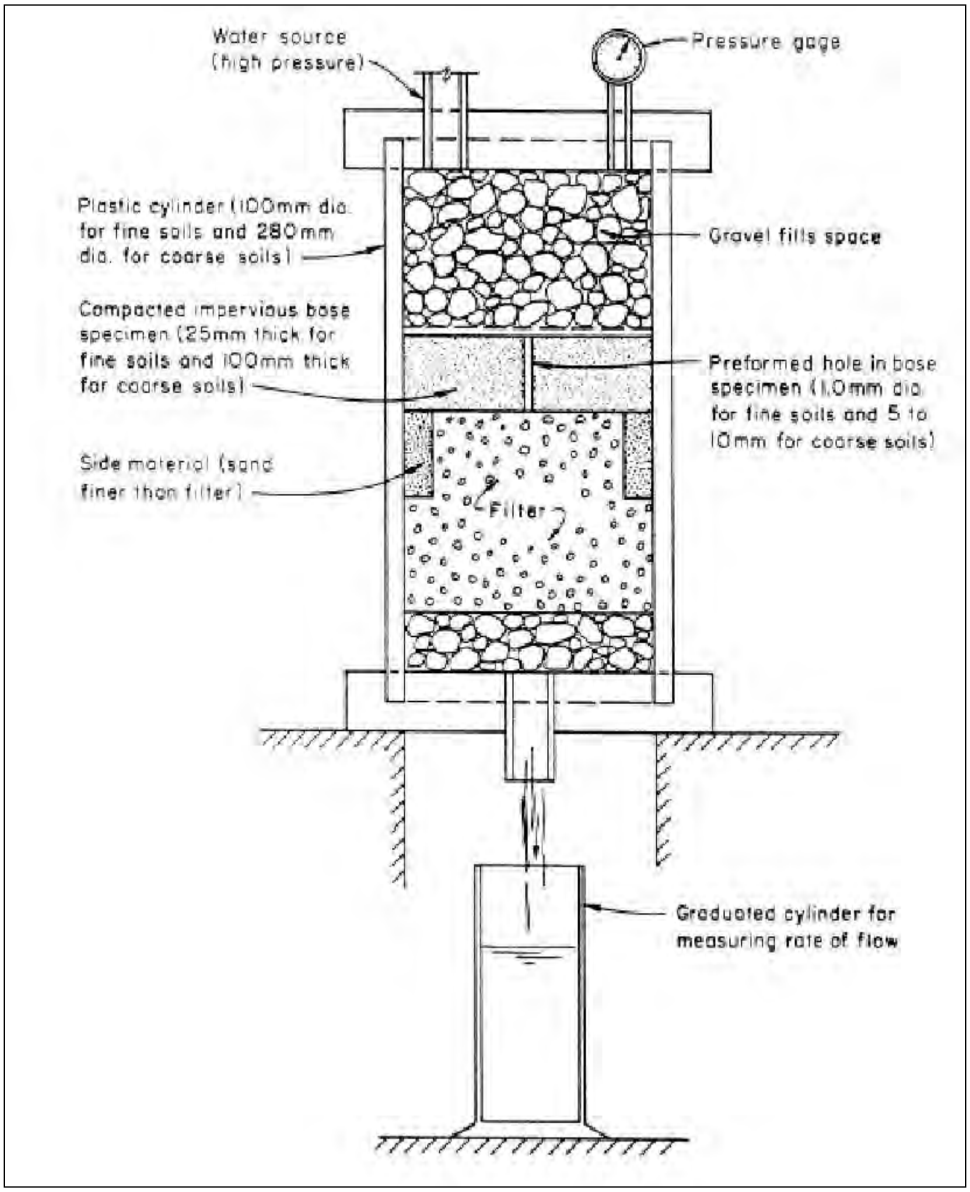

Figure F-2. NRCS No erosion filter test details (no-scale schematic).

6. The 3/8-in. to 1/2-in. gravels placed on top of the base specimen are only for the purpose of filling the space and have no influence on the test.
7. In the standard test procedure, the first step, after placing the cylinder with prepared base and filter in the testing stand, is to fill the space above the base specimen with water, forcing out the air. During this activity, a little water may or may not be seen coming out of the bottom.
8. After the top of the cylinder is saturated, the hoses to the water pressure source and to the pressure gage are connected, and the valve from the

water pressure source was cracked open an amount to give an initial pressure of about 10 lb/in.^{2} (psi).

9. This initial pressure is commonly held for about 1 to 2 min, recording the rate and appearance of the water flow emerging from the bottom.
10. Then, after the initial minute or two, the valve is opened completely, placing full reservoir pressure (about 60 psi) on the upstream end of the apparatus.
11. For tests on dispersive clays, the water source was distilled water with pressure supplied by compressed air.
12. The criterion used was that in tests judged "successful," the filter should be sealed with no visible erosion of the initial 1.0 mm diam of the hole through the specimen. For the typical successful test, the first flow emerged from the bottom soon after application of water under pressure at the top. The amount of the initial flow commonly was within the range from 1 to 15 ml/sec, occasionally more. The initial discharge was always colored with eroded clay particles in suspension, leaving no doubt that eroded particles of the base specimen were carried into the filter. During the last part of the test, the discharge gradually cleared until at the end when the flow was completely clear. The measured discharge quantity for the typical successful tests commonly varied over the 10- to 15-min test duration according to one of the following patterns:
    a. Remained at a more or less constant rate
    b. Decreased and then flowed nearly constant rate
    c. Increased and then flowed nearly constant rate
    d. Increased for a few minutes, then decreased and finally flowed at a nearly constant rate.

Test for measuring compressive strength of minus No. 4 fraction

1. Screen out all plus No. 4 sieve particles using a standard size sieve.
2. Using about 2,000 grams of dry soil, moisten the soil to 10% water content by adding 200 grams of distilled water to the 2,000 grams of dry soil.
3. Thoroughly mix the water and sand in a plastic bag and allow the sand and water to equilibrate for at least 1 hour.
4. Use the moist sand to compact a test specimen into a compaction mold similar to that used in ASTM D698A. Mold the sample in three lifts using Standard Proctor energy as described in ASTM D698A.
5. Compact the material to the density that will be required in the specification paragraphs.

6. A plastic liner may be used in the mold to ensure separation of the sand from the mold when disassembled.

Figure F-3. No erosion filter testing being conducted in Soil Conservation Service laboratory in Lincoln, Nebraska. (Photo credit – James Talbot.)

7. Mold the sample so that the mold is filled completely where a porous disc or other suitable extrusion shim can be placed on the compacted sample. Carefully extrude the sample with the compacted sand resting on the extrusion shim.
8. Place the molded sample on the porous stone or shim in a curing room with a temperature between 95 and 120°F for a minimum of 48 hr.

Figure F-4. No erosion filter test apparatus in Soil Conservation Service Laboratory in Lincoln, Nebraska. (Photo credit – James Talbot).

9. Perform a Compressive Strength Test of the cylindrical sample in accordance with ASTM D2166 at a strain rate of 0.5% per minute.
10. Perform tests on three samples (replicates). If the test results vary more than 30% from the average, test three additional samples.
11. Report the result as the average of the three tests.

Attachment G – Selecting Filter Gradation Band within Design Limits (Reclamation)

The design procedure results in minimum and maximum limits (control points) for grain size distribution based on particle retention and permeability requirements. These limits allow flexibility in the final stage of filter gradation selection based on the intended purpose of the filter. This section will describe how to select a gradation band within these limits for filters in different applications. For purposes of this example, it is assumed that the base soil is the same for each of these applications. The examples are based on a Category 2 base soil, and differs from the illustration used in Chapter 5.

Using the filter design procedure described in Chapter 5, the limits for this example Category 2 material are found, and the results are shown in Figure G-1. These limits can be thought of as the range in which filter gradation candidates can be entered. Filter gradation candidates within these limits will meet criteria for permeability (minimum limit) and particle retention (maximum limit). Depending on the planned use for a candidate, the gradation can be anywhere within this range and still meet these criteria. The next sections present several examples. **These gradations are presented as examples and should not be used for the applications described without going through the entire design procedure as described in Chapter 5.**

In general, the method of selecting the gradation band inside the limits can be done in three steps:

1. Begin with the smaller grain sizes since this is where the particle retention and permeability constraints are located (points A, B, and I). If particle retention is the more critical criteria, the gradation should be set closer to point B. If the permeability criteria are more important, the gradation band should be closer to point A.
2. Adjust the sliding bar GH based on the amount of uniformity that is desired in the gradation. If a more uniform gradation is desired, move the bar to the right, near point F. If a more broadly gradation is desired, move the bar to the left, near point E.

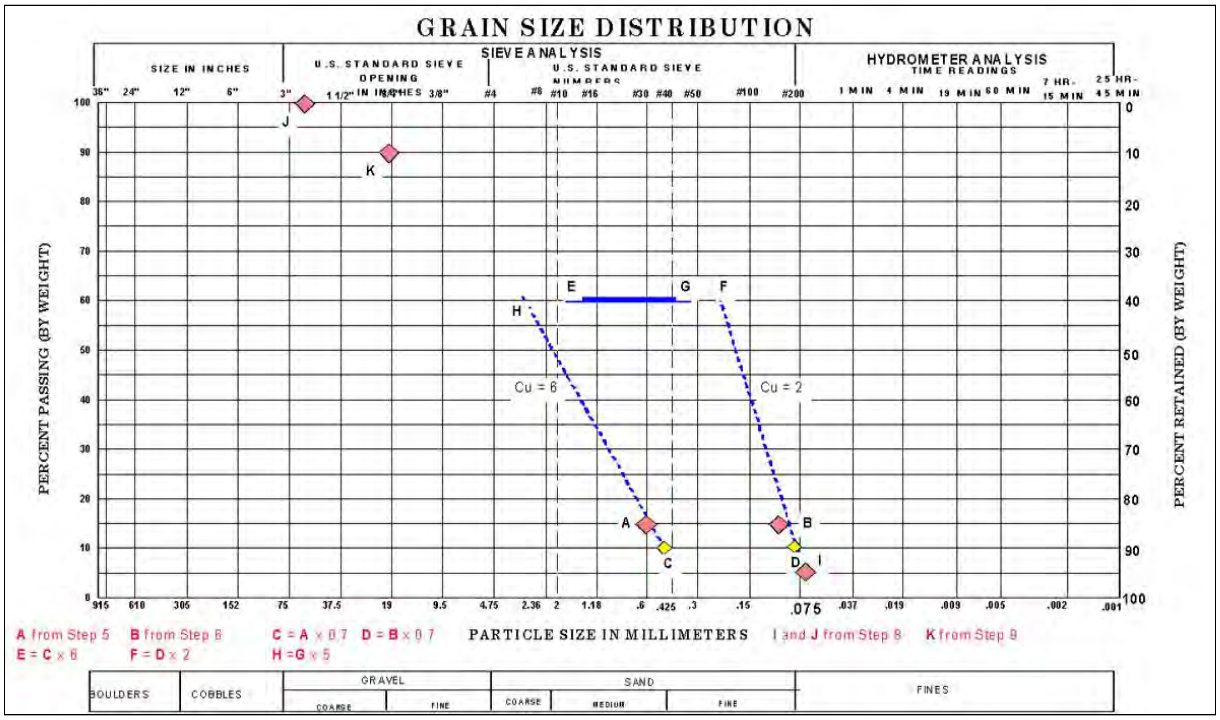

Figure G-1. Limits (control points) for an example Category 2 base soil.

3. Select the gradation range for the largest grain sizes. This portion of the gradation band has the least amount of constraints on it (only points J and K) and offers the most flexibility in the gradation selection. In general, the gradation bands should have the same or slightly flatter slopes than what is seen in the range of 30 to 60 passing. The gradation should also curve to the left similar to the relationship seen between points J and K.

Particle retention filter

In situations where particle retention is of the greatest interest, a filter gradation shown in Figure G-2 can be used. This filter gradation could be used for a chimney filter in which protection of the core is the primary concern. It could also be used for toe drains in which large amounts of seepage are not expected. Notice that this filter gradation is intentionally uniformly graded to minimize segregation potential.

The gradation was set by first selecting the finer side of the gradation band near point B. Next, since a more uniform gradation is desired, sliding bar GH is set to the right, near point F. The gradation is extended to pass through the sliding bar and finished by decreasing the slope and curving

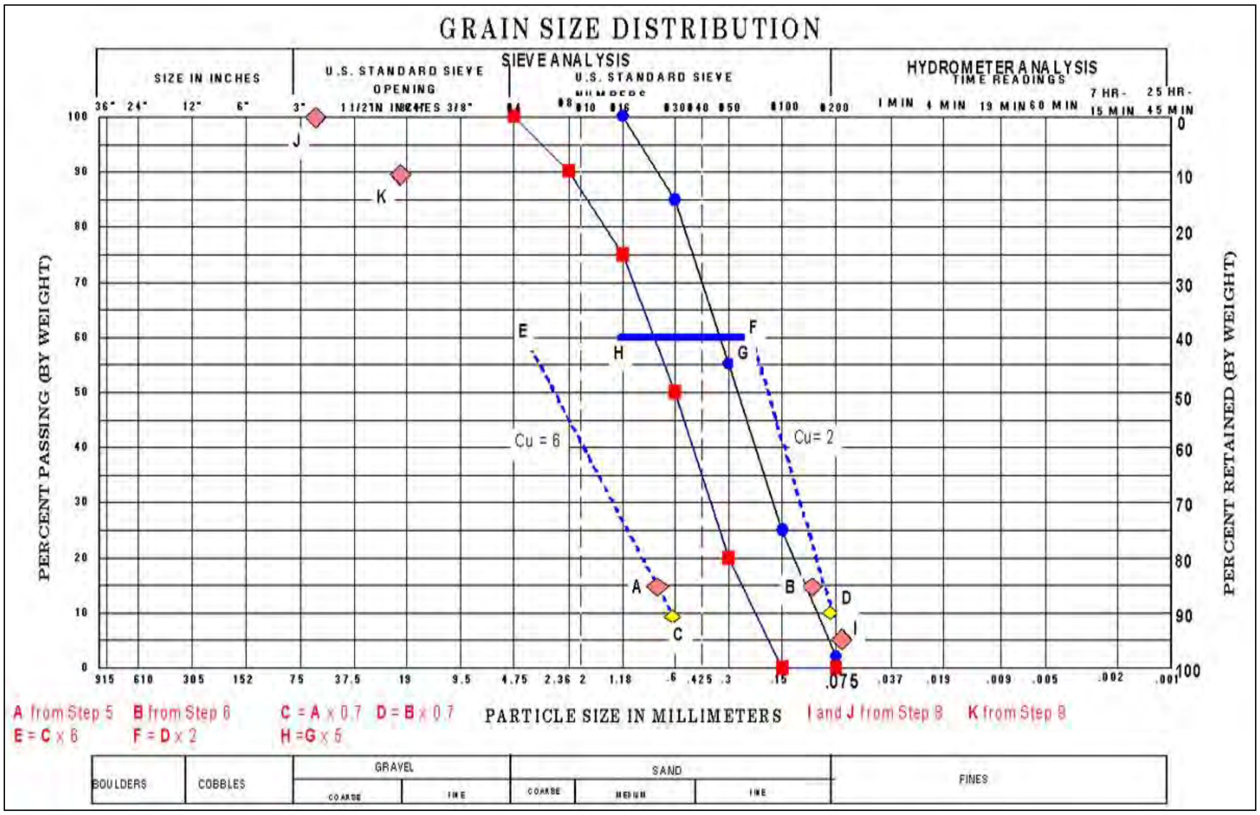

Figure G-2. Example particle retention filter gradation.

the gradation for the coarsest portion of the gradation. The resulting filter is a fine to medium sand.

Drainage filter

For cases in which drainage is the primary goal, such as toe drains on pervious foundations, the filter gradation shown on Figure G-3 can be used. In this example the D_{15} gradation is set near the upper limit of particle size (point A) to maximize the permeability of this candidate. To enhance the permeability characteristics of this candidate, the gradation is more uniformly graded where the slope of the gradation is about $C_u = 2$. To meet this slope, the sliding bar GH is set halfway between points E and F. The remainder of the curve is set to a slope slightly less than $C_u = 2$ and curves to the left. As mentioned earlier, this type of gradation could be used in toe drains on pervious foundations and also blanket drains on similar foundations.

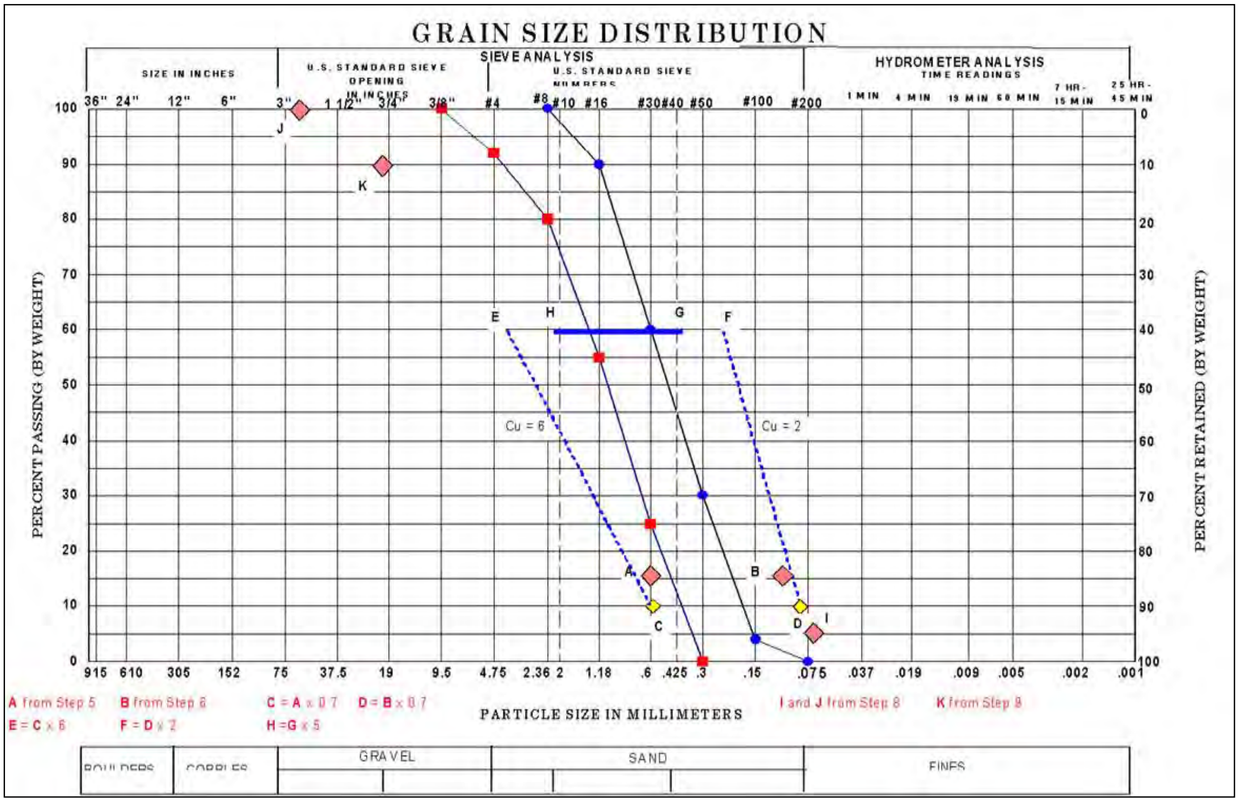

Figure G-3. Example drainage filter gradation.

Transition zone filter

Figure G-4 illustrates a broadly graded material that could be used as a chimney transition zone. The advantage of this gradation is in the economy of production since a wider range of grain sizes are used for a single zone. Note that while the candidate gradation defers to the particle retention criteria (set to the minimum D_{15} limit near point B), the coarser sizes are set the maximum limit (points J and K), hence spanning the entire range within the limits. This candidate has a coarser upper end than more uniformly graded candidates, permitting a minimum 1-in. material for the next transition zone. The reduction in the number of zones also results in a lower cost. While this gradation is more susceptible to segregation than more uniformly graded material, that amount of segregation is manageable using the construction techniques described in this manual.

Standard material filter

Figure G-5 illustrates the use of C-33 concrete sand, which is commercially available in most areas. This gradation was plotted as a check to see if it

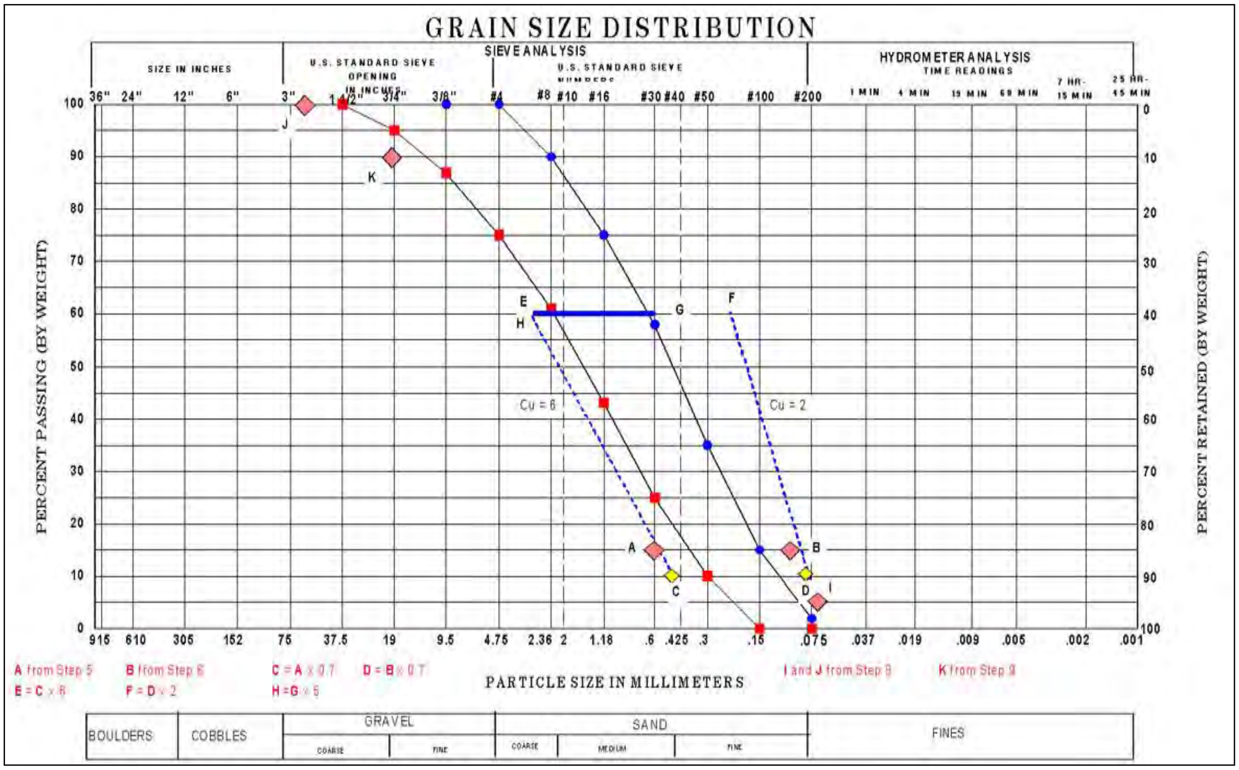

Figure G-4. Example transition zone gradation.

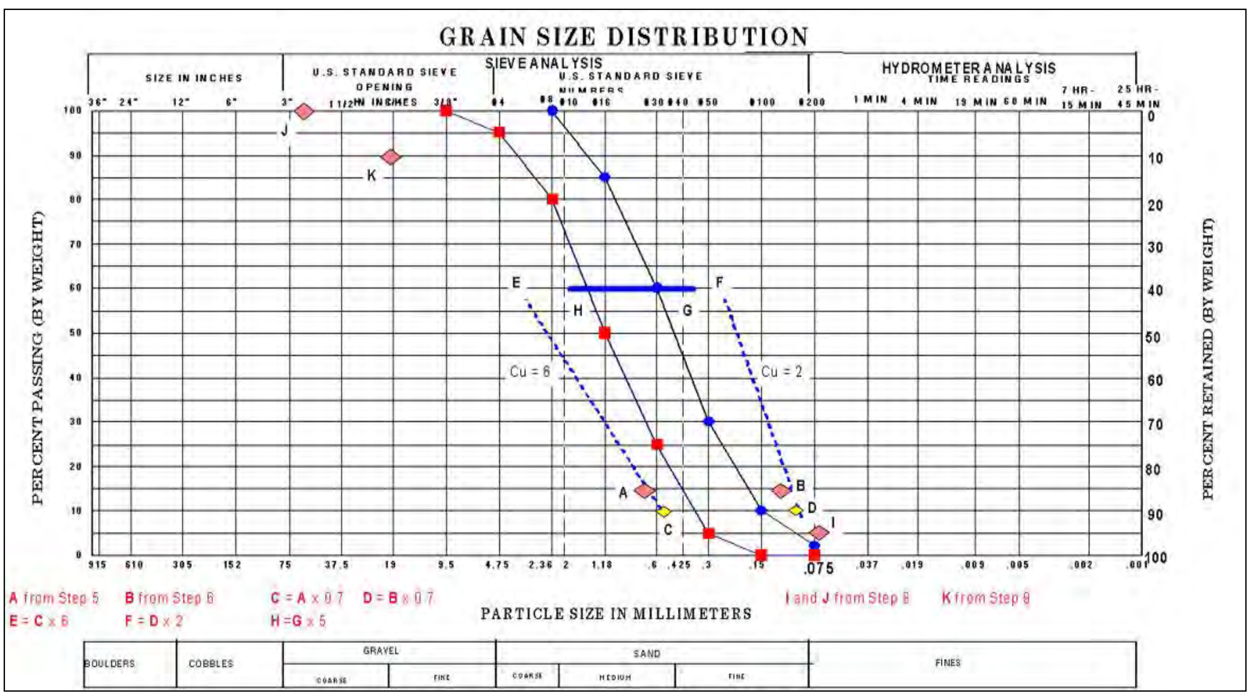

Figure G-5. Example standard material (C-33, concrete sand).

fell within the limits derived from the design procedure. Since it is within the limits, this material would be acceptable for use in this Category 2 base soil.

Link_001_Overview

Karl Terzaghi is generally credited with developing the first empirical guidelines for filter design. Fannin (2008) presents a thorough history of Terzaghi's studies of filter theory in the 2008 ASCE Terzaghi Lecture. Terzaghi's criteria for filter sizing to prevent particle movement from seepage were largely empirically based on laboratory studies. Others have examined the potential for particle movement on the basis of the sizes of spheres that are able to move through a matrix of particles of differing sizes.

Research by Sherard et al. (1984) and Foster and Fell (2000) has shown that the measurable property of the filter that best defines its function for stopping soil particles from entering is the D_{15} of the filter. The D_{15} of a filter is the size of a particle in millimeters of which 15% of the soil is finer and 85% is coarser than that size. It is obtained by plotting a grain-size distribution curve and interpolating the curve at the 15% finer size. Modern research indicates that the effective pore diameter of a filter (diameter of pores that will allow soil particles to pass) is about 0.11 times the D_{15} of the filter. Research also has shown that the best correlation between successful filter function in retaining soil particles was obtained by comparing the D_{15} of the filter with the d_{85} (the particle diameter at 85% passing) of the base soil or the soil being protected by the filter. This correlation between D_{15} and d_{85} was also noted in the filter design criteria proposed by Terzaghi in 1922.

The conclusions of recent filter studies show that the cross-sectional area of a continuous pore channel through the filter is irregular. The pore channel changes rapidly and repetitively from maximum to minimum size over a short distance. The pore channel's linear dimensions normal to the direction of flow vary from about 0.10 to 0.60 × D_{15}. By the time seeping water has traveled in the pores a relatively short distance such as about 5 × D_{15}, it has already passed through all the combinations of pore channel sizes and shapes. All pore channels in a given filter gradation have very nearly the same minimum dimension that will block particles and prevent them from passing through the filter.

The Sherard studies showed that particles smaller than about 0.10 × D_{15} that are carried in water suspension to the filter face will generally pass into, through the voids, and out of the filter. Particles larger than about

$0.12 \times D_{15}$ will be retained near the filter face. Thus, the filter acts like a laboratory sieve with openings about $0.11 \times D_{15}$ wide.

Proper filter design is to set the D_{15} of the filter at some multiple of the d_{85} of the base soil (for example, five to nine times depending on the type of soil). Because most soils have a range of particle sizes, the coarser particles are caught at the filter face first, and subsequent finer particles cannot pass through the coarser particles. When soil in suspension (turbid or sediment laden water) arrives at the filter face, the first very small colloidal clay particles will enter the filter and pass on through. The very fine sand or silt size particles that are larger than $0.11 \times D_{15}$ will be caught at the filter face. Subsequent colloidal or very fine silt particles will then be caught because the first particles caught at the filter face fill up the minimum void dimension, making the opening too small to pass the smaller particles. The result is that a cake of soil that gets finer as the buildup progresses is formed on the face of the filter. This filter cake can have a very low permeability because of the nature of its formation with finer particles near its surface. Soil in suspension can arrive at a filter face when the base soil has a crack or other opening in which water passing erodes the sides of the crack or when the filter is not in intimate contact with the base soil, allowing particles to move with flowing water.

Link_002_Bohio Dam Discussions (1902)

A large dam planned in conjunction with the Panama Canal project was the Bohio Dam. It was planned to block the Chargres River. The challenges at the site included a deep alluvial foundation of sand and gravel. Considerable discussion was presented in testimony before the Congress on how to protect against underseepage. The discussions on the project are some of the first that refer to underseepage and piping problems on alluvial foundations. The following sections have more detail on these discussions and similar projects that were compared to this one.

Reference: *Proceedings of ASCE, Volume XXVIII, January 1902.*

Discussion by Haines

One of the earliest known references to the use of filters is in testimony before a Senate committee by Colonel Hains, the head of engineering for the U.S. Army Corps of Engineers (U.S. Army Corps of Engineers 1902).

The following exchange took place in his questioning on the preliminary design of the Bohio Dam:

> ***The Chairman:*** *Is it not quite probable that a 90-foot head of water there would be sufficient in its weight and power to convert that seepage into a tunnel?*
>
> ***Colonel Hains:*** *I think not.*
>
> ***The Chairman:*** *Why? What would be the resistance that would prevent it?*
>
> ***Colonel Hains:*** *The weight of the superincumbent mass above it. Filters, you know, are constructed somewhat on that plan—filtering plants for water for cities.*
>
> ***Senator Kittredge:*** *Do I understand, General, that you think the seepage would at some time stop?*
>
> ***Colonel Hains:*** *It is believed that it would stop, on account of the light material which passes through the sand. You see, it necessarily passes through the sand with a very low velocity, and there is always a certain amount of matter that is in water on such streams as the Chagres, and that will be washed into these spaces and gradually fill them up.*

Discussion by Stearns

Additional discussion of the use of filters in embankment designs is provided in a discussion article written by Frederic Stearns for the American Society of Civil Engineers Transactions in 1902 (Stearns 1902). Stearns was contrasting the design of an embankment he designed for the Wachusetts Reservoir with another similar one done by another engineer, William Morison, for the Bohio Dam. He described a zone of gravel at the downstream toe of the dam that was included for filtering flow under the embankment as follows:

> *At the extreme downstream toe of the dam, an embankment of very coarse gravel was formed. Its maximum section is 19 ft high and 115 ft wide at the base. This embankment is*

> *for the purpose of permitting any water which may find its way through the dike to escape without the sloughing of the material. If the quantity filtering should prove to be large, there is a coarse gravel stratum below the surface which could be tapped by means of driven wells to furnish an escape for water filtering through that stratum, but it is not anticipated that these wells will be needed.*

In his paper, Stearns discussed experiments performed to evaluate the filtering mechanism between the base soils in the embankment and sands as follows:

> *Will Filtration Cause a Fine Material to Penetrate a Coarse One? - The question having been raised as to whether soil filled into a sand and gravel trench might, when the pressure was applied, penetrate the interstices of the sand, it was decided to make the following experiment, notwithstanding the fact that it is well known, from practical experience with water filtration, that fine particles are not carried to any considerable extent into the interstices of the sand. The 10-in. cylinder, already referred to, was filled to within 3 in. of the top with medium sand and then 3 in. of soil were put on the top; the end of the cylinder was close, and water was applied, with a pressure of 65 lb/in.^{2}. The water was allowed to filter through the soil and sand for a week, and upon examination at the end of this time there was no indication of any movement of the soil or that any of it had worked into the sand.*

In a later section of his discussion, Stearns discusses the gravel berm placed at the toe of the embankment:

> *If a large amount of water should filter through the material under the dam and come out at the lower end, it may cause some movement of material in the vicinity of the toe, unless precautions are taken to prevent it.*

Link_003_Terzaghi (1925)

One of the first proponents of the use of a filter to protect against backward erosion piping was Dr. Karl Terzaghi (Terzaghi 1925). In a 1925 publication he provided the following discussion of a graded filter to protect against piping at the toe of a concrete weir:

> *Foundations of Weirs on Permeable Ground. – This problem is one of the most difficult the foundation engineer has to deal with, because it represents a problem of bearing capacity complicated by the hydrodynamic pressure exerted by the seepage water. . .When flowing under head through permeable ground, the water exerts on the soil a pressure. In every point of the underground the pressure acts in the direction of the flow, and its intensity is directly proportional to the local hydraulic gradient. . .. The only efficient remedy is this: to keep the soil in the danger zone down, by means of a graded filter. The filter safeguards the free escape of the seepage water, and at the same time it prevents the soil particles from drifting away.*

Link_004_Harza

Following Terzaghi's early work, Harza (1934) discussed the use of drainage in embankments as follows:

> *As in the case of earth dams there are two schools of thought with reference to drainage, those generally for, and those generally against, it. The writer believes that it provides a useful factor of safety especially in uncertain ground. He believes in the principle of imposing effective resistance to seepage through such a distance as is required until the quantity of leakage is sufficiently reduced, and then providing ample artificial, permanent, and well-constructed drainage works which will relieve as much as possible through a controlled route where its escape will be easier than through the material and where it can do no harm. Only the escape of seepage through the material is hazardous and it is better to provide a route through which it can escape more freely that through the material itself. It*

must be kept in mind that seepage will follow all routes of escape in inverse proportion to their resistances; and drainage, therefore, becomes a safety factor, not an elimination of toe-flotation possibilities.

Thus, any pervious stratum or lense that may conduct excessive pressure beneath a dam, possibly to burst upward into tail-water, should be intercepted or blanketed near the heel where possible; and then near the toe it should be drained as freely as possible. Water will not burst upward through the sand from internal pressure if there is an easier route of escape than by lifting the sand.

This line of escape may be created by well-points, screened driven wells, or intercepting trench if necessary to gain capacity, or (if local) by a dug well. Drainage wells can be connected by a header or filter gallery under or in the apron or toe of the dam and carried to a pump sump if it is desirable to hold the drained level below tail-water. This need might readily arise to protect an apron against uplift ahead of the hydraulic jump. Water would then drain toward the filter gallery from both head-water and tail water. A complete line of drainage along the down-stream face of the-toe sheeting could be used to eliminate most of the upflow through the material itself in an unstable situation requiring such precaution. Drainage is a useful tool for the designer and is capable of a great variety of useful applications.

Harza's recommendations for protecting the toe of the dam against flotation and piping are as follows:

Safety from toe flotation is best promoted:

(a) By choosing a design with a depressed toe or toe cut-off and one which dissipates head rapidly along the early part of its route, leaving as little remaining head as possible to be lost during the upward flow into tail water;

(b) By supplying a special fill down stream from the toe, an inverted filter in principle;

(c) By providing an inverted filter drain under the toe of the structure itself, ahead of the toe cut-off; and

(d) By means of wells, drainage galleries, etc., at the toe.

Discussion by Justin

In a discussion of Harza's paper, Justin recommended the following design for a dam on a pervious foundation.

> *In cases where the escape of a large amount of seepage at the toes is anticipated, some kind of drainage system is indicated with the drains partly clogged by gravel and stone so as to form inverted filters, thus preventing any of the foundation material from being carried through. Heavy rock fills in the down-stream part of an earth dam with the foundation protected by an inverted filter may perform this function.*

Discussion by Terzaghi

Terzaghi (1934) discussed Harza's paper in part as follows:

> *The results of experiments with perfectly homogeneous sands showed that the escape gradient at which piping occurred is practically equal to the flotation gradient.*
>
> *In most cases the failure of dams begins with the formation of springs which discharge a mixture of soil and water. For this reason, as soon as the writer recognized the importance of the flotation gradient, he concentrated his attention on the practical possibilities connected with reversed filters.*
>
> *The second requirement to be satisfied by a reverse filter is that the voids of the filter must be small enough to prevent a loss of soil without interfering appreciably with the discharge of the water. In his earlier designs, the writer speci-*

> *fied that the average diameter of the grains of the lowest layer of the filter should be approximately five times as large as that of the largest grains of the covered soil.*
>
> *However, as his experience increased, he found it advisable to determine, experimentally, in every case the most satisfactory grain size for the bottom layer of the filter. Each one of the succeeding layers is about five times coarser than the preceding one.*

In his discussion of the Harza paper, Terzaghi referred to patents he filed in Austria in the 1920s for filter zones downstream of concrete gravity dams. The discussion included drawings of the patents. The drawings illustrate what Terzaghi considered an essential requirement for an effective filter, which is to prevent the flotation of the covered soil by using a superimposed load or a rigid structure with weight sufficient to overcome the upward forces.

Link_005_Bertram (1940)

Bertram's studies (Bertram, 1940) on the properties of filters examined what gradation of filter would be required to prevent the movement of base soils that were uniform sizes of Ottawa sand and crushed quartz. In modern filter design nomenclature, the base soils studied were all in Category 4, as described in Chapter 3. These soils have less than 15% finer than the No. 200 sieve. Bertram concluded that significant amounts of the sandy base soil would not penetrate into a coarser filter zone so long as the 15% size of the filter material was not more than 8 to 10 times the 85% size of the fine base material. Results were the same whether flow was upward or downward and regardless of the magnitude of the hydraulic gradient.

Bertram also recommended that filter gradation curves be approximately parallel to base soil curves. Other experiments were performed with a filter in contact with a plate having circular openings. Those results led Bertram to conclude that to prevent loss of filters into openings required two filters—a coarse filter next to the openings and a finer one outside.

Link_006_U.S. Army Corps of Engineers (1941)

This article (U.S. Army Corps of Engineers 1941) followed up on Bertram's experiments and included additional tests using base soils and filters that were different than those used in Bertram's experiments. Studies of filtering were carried out by placing a thin layer of base soil on a filter sand. The plan was to vary the coarseness of the filter until a failure of the base soil occurred from piping. Because Bertram's studies only used sand, this study was intended to include more problematic soils such as coarse silts and very fine sands.

Interesting results were noted for the first two trials. A loess soil with most of the particles between 0.05 millimeter (mm) and 0.005 mm was tested first. "The material proved to be so impervious that the velocity with which water would pass through it was not sufficient to produce any large movement of particles." Also too impervious was a sandy loam, containing particles from 1.00 mm down to clay size 0.005 mm.

The final material used as a base soil was a poorly graded fine sand that ranged in particle size from about 0.3 mm down to 0.05 mm, or from about a No. 48 mesh down to a No. 200 mesh, with a coefficient of uniformity value of 1.75. Experiments showed no failure so long as ratio of D_{15} to d_{85} was 5 or less.

The primary conclusions from this investigation were:

1. A fine material will not wash through a filter material if the 15% size of the filter material is less than five times the 85% size of the fine base material. This criterion was in contrast to Bertram's findings that the ratio of D_{15} to d_{85} could be in the range of 8 to 10.
2. In addition to meeting the above size specification, the grain size curves for filter and base materials should be approximately parallel in order to minimize washing of the fine base material into the filter material.
3. Filter materials should be packed densely in order to reduce the possibility of any change in the gradation due to movement of the fines.
4. A filter material is no more likely to fail when flow is in an upward direction than otherwise, unless the seepage pressure becomes sufficient to cause flotation or a "quick" condition of the filter.
5. A well-graded filter material is less susceptible to running through the drain pipe openings than a uniform material of the same average size.

However, even a filter material having a wide range of gradation cannot be used successfully over a drain pipe having large openings since enough fine particles to cause serious clogging will move out of the well-graded material into the pipe.

Link_007_Bureau of Reclamation (1955)

The research that formed the basis for the Bureau of Reclamation (Reclamation 1955) criteria was undertaken when concrete linings below check structures on the Friant-Kern Canal in California failed. Filter materials surrounding drainpipes under the structures were removed and found to have met commonly used Terzaghi criteria relating the filter D_{15} size to the base soil d_{85} size.

In their experiments, various combinations of filter materials were tested against various base materials at applied heads of 2.5 to 30 ft. Filters studied were both uniformly graded and broadly graded.

A summary of the conclusions from that research are summarized as follows:

1. For uniformly graded filters, the ratio of the filter's D_{50} size should be from 5 to 10 times the d_{50} of the base material for uniformly graded filters.
2. For more well-graded filters, the ratio of the D_{50} to the d_{50} should be between 12 and 58.
3. For well-graded filters, the filter D_{15} should be from 12 to 40 times the d_{15} of the base soil.
4. The filter should have a maximum particle size of 3 in., and it must have no more than 5% finer than the No. 200 sieve.
5. The gradation curve for the filter should be about parallel to the gradation curve of the base material in the range of finer sizes.
6. Perforations in pipe should be smaller than one-half of the surrounding filter d_{85} size.
7. Research showed that filters derived from crushed rock required a finer gradation to protect against piping of a given base material than did rounded filters. For crushed rock filters, the ratio between the filter's D_{50} size to the base soil's d_{50} size was required to be between 9 and 30, and the ratio between the filter's D_{15} and the base soil's d_{15} was required to be between 6 and 18.

An important finding of their study that was not routinely followed in several decades following their research was that base soils should be re-graded on the No. 4 sieve before designing a filter to be compatible with them. This basic principle was not emphasized again until Sherard's research on filter testing in the 1980s.

Reclamation criterion that used the 50% finer sizes for design was not widely used after the publication of the Sherard articles and was supplanted by other criterion for designs in following years.

Link_008_Kassif

Kassif et al. (Kassif 1965) researched the filter failure boundaries for clays. In their tests, clays of relatively high plasticity were compacted in a device where the specimen rested on relatively coarse gravel with D_{15} sizes of 2 to 4 millimeters (mm). Failure of the specimens did not occur even though gradients of up to 50 were applied to the specimens, and the tests were left at this pressure for two months. A gradient of 50 implies an applied pressure of 125 centimeters (cm) of water, which is equivalent to 4.17 ft of water acting on a specimen that was only 2.5 cm thick.

Other tests were performed with the clays resting on a Lucite plate in which 15-mm- (0.6-in.-) diam holes were drilled. No failures of the specimens occurred when a surcharge (confining pressure) of 0.2 kilogram per square centimeter (400 pounds per square foot) was applied to the specimen, even when a water pressure was applied exerting a gradient of up to 1,000. A gradient of 1,000 translates to an applied pressure of slightly over 40 pounds per square inch applied to the top of a 3.0-cm-thick specimen.

In summary, Kassif concluded that soils with significant plasticity have very different filter requirements than sands when considering strictly intergranular seepage flows. This is true because the electrochemical bonds in the clays plus their low permeability create a condition where a relatively high discharge hydraulic gradient is required to detach particles and initiate backward erosion piping. In an extension of this work by Kassif, Dunnigan suggested (Dunnigan 1988) that based on the research conducted with Sherard, that

> *It is not necessary to apply quantitative criteria to filters used for protecting low permeability fine grained soils that*

can never be subjected to a concentrated leak and where the gradient is low. Non critical filters for low permeability fined grained soils may consist of any uniformly graded sand or sand gravel mixture.

Link_009_Vaughan and Soares (1982)

In a 1982 article, Vaughan introduced the concept of the "perfect filter." The article was written on the basis of a study of sinkholes that developed in a dam constructed in Great Britain that was completed in 1964. The dam had a fairly broadly graded filter next to a core zone. Based on his study of the problems at Balderhead, Vaughan recommended designing a filter that would be effective in retaining the smallest particles that might be eroded from the core of a dam. The recommendation was based on sinkholes that developed on an embankment with a fairly broadly graded filter zone.

The definition of a perfect filter from Vaughan's article is, "The design principle adopted was to define a 'perfect' filter as one which will retain the smallest particles that can arise during erosion even if they arrive at the filter interface after complete segregation, unaccompanied by larger particles which would allow self-filtering to occur."

A large sinkhole appeared in the crest of the dam three years following construction. The sinkhole was attributed to hydraulic fracture of core material and internal erosion that occurred slowly over a period of 14 months. The narrow core had between 35 and 65% fines. The material was a well-graded glacial till with clay fines. The filter design was partly empirical but also based on the ratio of the D_{15} of the filter to the d_{85} of the base soil. However, no correction for gravel content was made, and the filter design was quite coarse, composed of crushed limestone. The designed filter had D_{15} sizes of between about 0.25 and 2.5 millimeters (mm), significantly larger than current design criterion would permit. Current design criterion would require a D_{15} size of no larger than 0.7 mm based on the re-graded core zone soils, which would fall in Category 2 in current terminology. At the time the article was written, re-grading base soils on the No. 4 sieve was not routine. Figure 1 shows the range of gradation of the core zone soils at Balderhead Dam after re-grading the soils on the No. 4 sieve. The figure also shows the designed filter gradation for the site.

Investigations showed that the installed filter was even slightly coarser than the design band. The actual filter zone had D_{15} sizes up to about 6 mm, whereas the filter design specified a maximum D_{15} size of about 2.3 mm. Using modern filter design methods, the maximum D_{15} size of a filter for the re-graded core zone soils would be about 0.7 mm. The conclusion is that the filters used in the drain then were about nine times larger than the 0.7 mm D_{15} size that would be used in current criteria.

The concept of a perfect filter was predicated where the finest particles that can erode are retained by a filter sufficiently fine grained to retain them. Vaughan's perfect filter theory assumed that no particles that are detached during the erosion of cracks in the base soil are available to seal the filter face. Sherard's later studies showed that eroding silt particles are effective in developing a filter cake at a filter face.

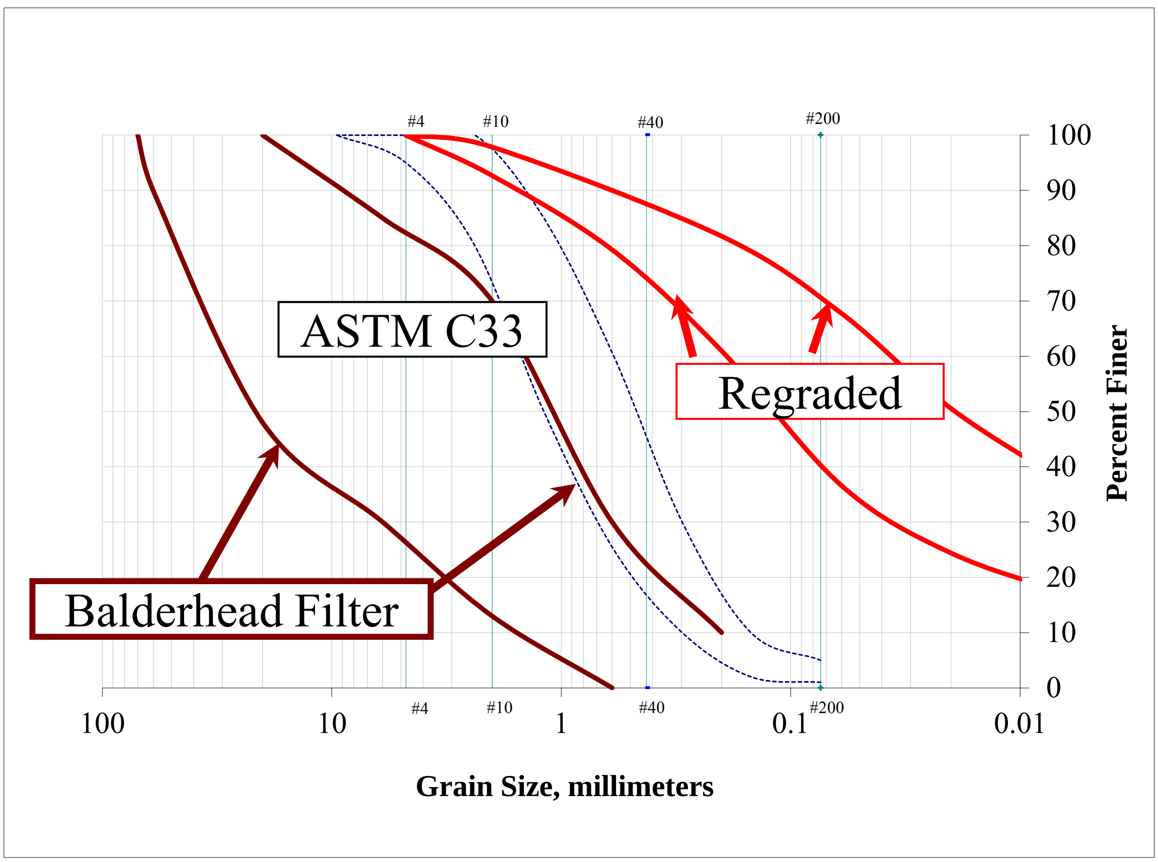

Figure L-9-1.—Re-graded core zone from Balderhead case history.

Vaughan's perfect filter theory used filter testing that was performed in a cylinder using a thin suspension of flocculated clay and candidate sands.

Failure was defined when flow was not attenuated by the filter. Based on the problems with the core at this site, a design philosophy was felt to be needed for similar dams. Vaughan warned against designing a filter with cohesion to the extent that the filter could sustain a crack, thus negating the effectiveness of the filter. If the temporarily cracked filter did not collapse with the introduction of the flow of water from a newly formed crack in the core, then fines could wash through the crack in the filter. Often, when designing a filter using the Vaughan procedure, the D_{15} size of the filter will be at the boundary between coarse silt and fine sand. For example, the D_{15} size at Carsington Dam in the UK, also described in the Vaughan article, ranged from 0.08 to 0.17 mm. This implies that much of the filter gradation below the D_{15} size is in the coarse silt range and, if cohesive, will sustain an open crack. Vaughan suggested the “sand castle test” to evaluate whether a filter with a substantial percentage of fines passing the No. 200 sieve, 0.074 mm, can exhibit cohesion and sustain a crack when inundated with water. Vaughan continues to recommend the use of permeability tests in a recent article (Vaughan et al. 2005).

Several discussers of the Vaughan article expressed their opinion that the lack of fine sand fraction in the Balderhead Dam was the primary reason the chimney zone did not provide satisfactory filtering. They further suggested that recommendations by Vaughan that resulted in adding a fines content of up to 15% to designed filters was ill-advised and would result in filters with unacceptable properties.

Disagreement exists on the concept of providing a perfect filter. The recommendation made in several discussions of Vaughan research is basically to continue using sand rich filters with D_{15} sizes in the range of 0.4 to 1 mm. Others continue to value the concept of the perfect filter. Following is a summary of several discusser’s opinions of the Vaughan recommendations.

Discussion by Ripley

As noted above, several engineers strongly differed with Vaughan on the use of finer filters to address the problems at Balderhead Dam. One of the most vocal opponents was C. F. Ripley, a consultant from Victoria, Canada, who stated his opinion as follows:

Serious internal erosion and malperformance of filters of central core rockfill dams have been reported not only at Balderhead Dam, but at dams in Norway and in Canada. In each case, the filter materials have had low sand content and have been so widely graded that significant segregation during normal handling procedures was inevitable. On the other hand, the writer has not found a single case of piping or internal erosion of core fines where the core was protected with a filter zone of clean cohesionless sand-rich material for which care was taken to prevent segregation during placement, and where necessary, the filter zone itself was adequately protected by appropriate downstream zones.

Discussion by Kleiner

Kleiner (2006), in a United States Committee on Large Dams conference article, provided an overview of filter research and tests done by a number of researchers. In the article, Kleiner discussed and contrasted the research from several prominent engineers. He summarized important differences in the recommendations of Vaughan and those of Sherard and others. The following discussions are taken from Kleiner's article:

The fundamental difference between the Vaughan and the Sherard/USDA SCS approaches to filter design is their individual concepts of what is required of the critical downstream filter. Vaughan's approach requires that the filter retain clay flocs without any self-filtering action whereas the Sherard/USDA SCS approach allows the erosion of silt-sized particles from the walls of a crack to clog the filter surface thus retaining fine silt and clay size particles through a self-filtering action. This difference is clearly demonstrated in the laboratory test procedures conducted by each researcher: the use of a slurry of clay flocs using river water and no dispersant in the Vaughan tests, and the use of the "no-erosion filter" test with a 1.0 mm preformed hole in the Sherard/USDA SCS tests.

In the author's opinion, the Vaughan approach is too conservative and can lead to a filter that is finer than required *for proper functioning. The reason for this is that silt sized particles are available in substantial quantities in all fine-grained clayey soils and, thus, are available to seal concentrated leaks that could result from core cracking. Hence, it is not necessary to provide a perfect filter to catch clay flocs of 5 or 10 microns diameter.*

It can be argued that any crack across the cohesionless filter will collapse in the presence of water and that self-healing will quickly occur even with a substantial percentage of coarse cohesionless silt particles. The Sherard/USDA SCS design criteria limits the D_{15} size to a minimum of 0.1 mm with no more than 5% of coarse cohesionless silt passing the No. 200 sieve. This eliminates any possibility of sympathetic cracking across the filter, and, from this perspective, in the author's opinion, is the more appropriate design procedure.

Additional discussions of factors that can affect the ability of a filter zone to sustain a crack are included in Chapter 5.

Link_010_Sherard (1984)

In the 1980s, the Soil Conservation Service (now the Natural Resource Conservation Service [NRCS]) contacted Dr. James Sherard to solicit his evaluation of a number of failures of their small earthen embankments that had failed upon first filling. Dr. Sherard studied the problem and reported his conclusions in an internal document in 1972. Subsequently, the SCS and Dr. Sherard embarked on an extensive investigation to determine whether existing filter criteria were satisfactory to design against internal erosion. Dunnigan (1988) provided a detailed history of the research effort Dr. Sherard and SCS conducted.

A series of seminal articles based on the research done in the SCS laboratory at Lincoln, NE, followed. The articles covered a wide variety of base soils that were studied. The articles resulted in a detailed group of recommendations for filter design that were subsequently incorporated into the SCS design memoranda. The first design criteria document published by SCS was Soil Mechanics Note SM-1 (1986).

Sherard became interested in a special problem involving broadly graded embankment soils at about the same time as his experiments with SCS were ongoing. He subsequently determined, based on field evidence and additional laboratory tests, that filter designs based on the total gradation curve of broadly graded soils resulted in overly coarse filters.

An important outgrowth of the Sherard studies was the now standard practice of re-grading broadly graded soils and designing a filter based on the finer fraction of the sample. Several approaches have been used to re-grade broadly graded samples, but the most common one is to re-grade on the No. 4 sieve.

Filters for sands

The research by Sherard was performed first on sand materials to establish the basic properties of sand and gravel filters (Sherard et al. 1989). Base sand soils consisting of uniform gradations (nearly all one size particles) of fine to very fine sand were placed over filters and water was run through the system to try and wash the sand particles into the filter. The gradation of the filter was made coarser and coarser until the sand particles began to wash into the filter. The point where sand began to wash into the filter was established for a range of sizes of base sands. The conclusion of the research was that so long as the D15 of the filter was less than about nine times the d_{85} of the base sand, a successful condition resulted. The ratio of $D_{15}/d_{85} = 9$ that defined a successful filter was consistent over a wide range of base soil sand gradations from very fine to coarse sands. As shown in Figure 1, the base sands studied had d_{85} values between about 0.1 millimeter (mm) and 2 mm. Terzaghi had proposed designing filters with the D_{15} equal to or less than five times the d_{85} of the base soil. The researcher's recommendation was to regard Terzaghi's criteria as being valid because they incorporated a safety factor of about 2.

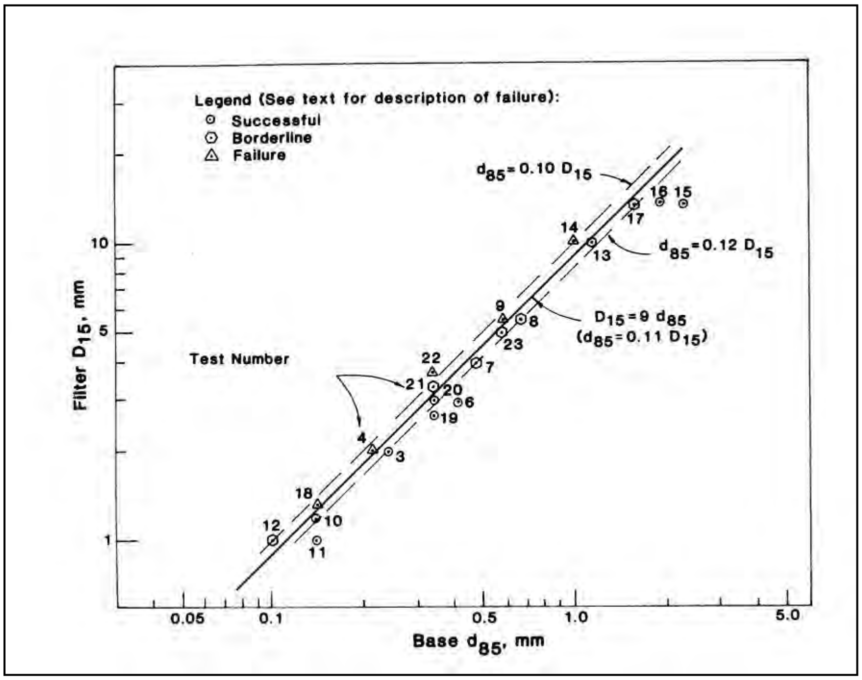

Figure L-10-1.—Relationship between D_{15} and d_{85} in initial SCS filter tests on sands and gravels.

Filters for silts and clays

The Sherard research (Sherard 1984b) then moved to silt and clay base soils. Laboratory experiments to investigate filters for silts and clays were begun after the research on sands and gravel base soils was completed. The 36 base soils ranged from nearly cohesionless silts to tough, highly plastic clays and included some highly dispersive sodium clays from dams that had failed by piping. The filters used were subrounded to rounded, alluvial sands, and sand gravel mixtures. The filters were carefully fabricated by combining known weights of carefully sieved materials, using sieve sizes which ensured that the D_{15} size was reliably known. A total of 25 different filters were used with D_{15} ranging from 0.3 to 9.5 mm.

In the experiments to determine the limits of filter compatibility for a variety of silt and clay base soils, the following experimental setup was initially used. A specimen of the base soil, a silt or clay, that was from 30 to 60 mm thick (about 1.2 to 2.4 in.) was compacted at about standard Proctor optimum water content on top of the filter being evaluated. Water pressure was then applied to the top of the base soil beginning with a pressure of 1 kilogram per square centimeter (kg/cm^2). A water pressure of 1 kg/cm^2

corresponds to hydraulic gradient across the base soil specimen of from 167 to 333, depending on the thickness of the specimen.

At these values of hydraulic gradients, no piping could be initiated even when the filter being evaluated was a relatively coarse gravel. The following quote is from the Sherard (1984b) article: "At relatively low pressures, generally below 1.0 kg/cm^2, no filter failures occurred, even for very coarse filter tests lasting many weeks. The small quantity of water seeping from the base sample into the filter had very little energy, and there was no tendency for the fine clay or silt base material to enter the filter pores." One set of tests was made using a clay of low plasticity, a soil with a liquid limit of 35 and a plasticity index of 11 and a d_{85} size of 0.039 mm. The following conclusion was stated in the 1984b article:

> *Conventional downward flowing filter tests were made with a compacted base specimen about 8 cm thick. No slot was formed in the base specimen. The filter was a compacted uniform gravel graded downward from 1-in. (25-mm) maximum diameter, and head was about 1 .5 m for a gradient of about 150/8 = 19. In these tests the tailwater level was kept above the bottom of the base specimen so that the filter-base interface was always saturated. The relation between filter and base soil particle sizes in these tests was far outside current accepted filter criteria, about as follows: D_{15}/d_{85} = 150; D_{50}/d_{50} = 410; and D_{15}/d_{15} = 1,360. Two separate tests were made with the same base and filter. One was continued for several months. The tests were stable. On dismantling the specimens at the end of the test, we found that the compacted silt had remained in a stiff state at the filter interface with no movement of silt particles into the filter. During the test a small flow of clear water emerged continuously (about 5 ml/hr).*

To induce failures in the base specimens, the water pressure was raised in 0.5-kg/cm^2 increments upwards to a pressure of 6 kg/cm^2, at which point, the hydraulic gradient was about 800 to 1,600:1. The specimen eventually developed a concentrated leak as described in the article:

> *These leaks appeared suddenly, and visual observations showed that they resulted from deformation of the specimen under water pressure, stress transfer, and hydraulic fracturing. After the initial leak developed, either the eroded material sealed the filter face and the leak stopped (successful filter) or it was carried through the filter without sealing (unsuccessful).*

For several reasons, this initial experimental setup needed improvement. The concentrated leak that developed from hydraulic fracture of the base specimen was irregular and unpredictable. The researchers determined that it would be advantageous to use a preformed crack in the base soil rather than creating a crack by hydraulic fracture. The effectiveness of the filter could be judged against the erosion of the preformed slot or hole in the base soil when the feature was eroded by the applied water pressure. The research developed a variety of slot tests and No Erosion Filter Tests to investigate the mechanism of a preferential flow path carrying eroded particles to a filter face. A simulated crack or opening was made in the compacted silt and/or clay running completely through the specimen where it encountered the filter.

The tests for silts and clays were made by introducing water at a pressure of 40 pounds per square in. (psi) to the upstream end of the soil specimen and allowing it to run through the simulated crack and into the filter. The 40-psi pressure was used simply because it was the city water supply pressure available at the laboratory where the testing was performed. It represents a head of water of approximately 92 ft. The first surge of water through the simulated crack came through the filter as a strong surge of water and was slightly cloudy.

For successful filters, the flow discharging from the apparatus dropped off rapidly so that, within 5 to 10 sec, flow had either stopped completely or reduced to a drip of clear water. Coarser filters were then used for the same soil until the flow did not stop but continued running more and more turbid until much of the soil specimen had washed through the filter. The experiments showed that the point at which a filter became too coarse and a failure occurred could be defined with a relatively close tolerance. That is, a given successful filter would become an unsuccessful filter by increasing the D_{15} size by only a slight amount. The boundary between success and failure had a wide range for the various silts and clays tested, but all failures occurred where the filter D_{15} was more than nine times the d_{85} of the base silt or clay. The tests demonstrated that a successful filter would stop the flow through a crack even under severe conditions.

Figure L-10-2 (Figure 1-2 from the reference document) shows some filter test results for silts and clays. Note that the plotted points define the ratio between the D_{15} and the d_{85} of the successful filter boundary for each of the soils tested. It shows that the ratio was no smaller than nine for this ratio for any of the silts or clays tested.

From observations of events during the testing and examination of the filter cake, it was postulated that the clogging of the filter progressed in the following manner: (1) of the first particles to reach the filter face, the colloidal particles (smaller than 0.002 mm) likely entered the filter and passed on through—hence the cloudy water that first emerged, (2) of the first particles to reach the filter face, the silt and sand size particles were caught, and (3) subsequent colloidal and other particles were caught on the sand and silt size particles first caught at the filter surface and no more soil particles entered the filter. This segregated layer system coupled with the high water pressure created a dense layer with very low permeability.

The testing demonstrated that a filter with a properly designed gradation prevented failure from internal erosion and caused the cracked zone to be healed so no cracking could be found when the process was complete. For filters in contact with the soil where there is no crack, the filter supports the discharge face with points of contact spaced at some distance determined by the gradation of the filter. Filters that were successful in catching

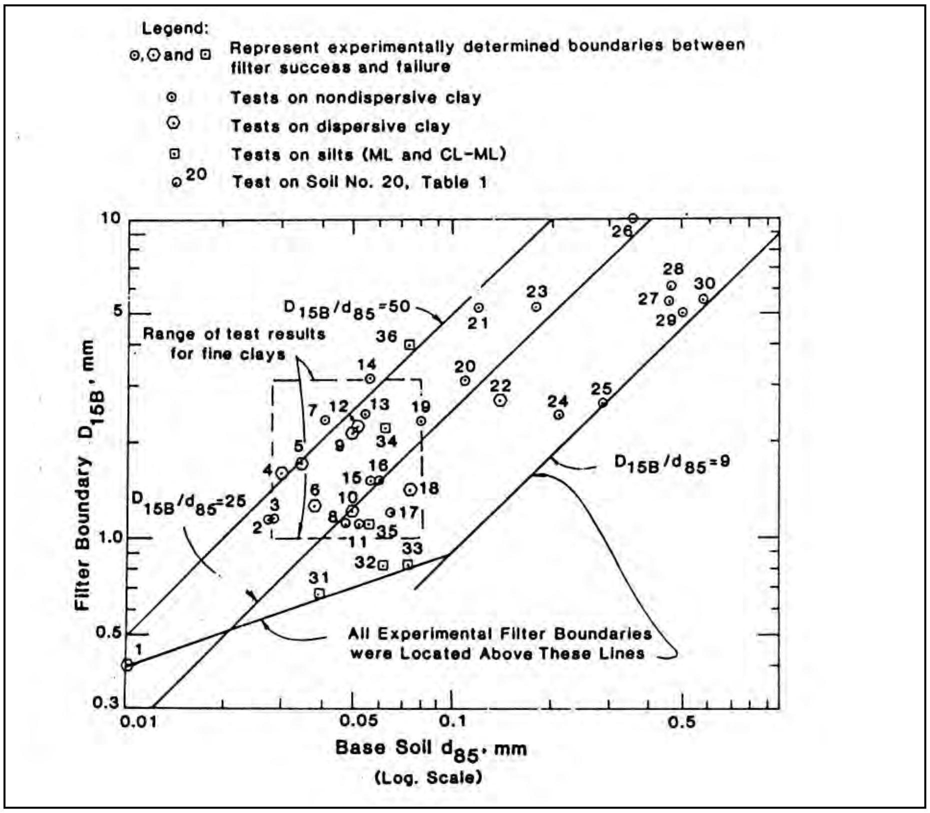

Figure L-10-2.—Summary of test results.

all the particles eroded from the sides of the cracks and openings appeared to be successful in preventing any particles from moving on the discharge face under high gradients where there was no crack. Apparently, there is some arching between the contact points where the filter is in contact with the discharge face to prevent any movement of particles. Coarser filters or other materials that do not support the discharge face with closely spaced contact points like the granular filters will not prevent soil particles from moving when the gradients exceed the critical gradient.

Chapter 1 includes more of the theoretical discussions of the ways that filters for silts and clays function.

The no erosion filter test

This test was developed during Sherard's research and continues to be a standard for site-specific filter compatibility studies. More detail is provided in Attachment F.

Part 633, Chapter 26 of the NRCS NEM

As noted previously, the first agency filter criteria published after Sherard's research was the SCS's Soil Mechanics Note SM-1 (1986). In October 1994, the guidance document Soils Mechanics Note SM-1 was revised and Chapter 26, Part 633 of the NRCS's National Engineering Manual was issued. The primary changes in the SCS approach to filter design that were included in Chapter 26 restricted the width and broadness of gradation of the designed filter. The intent was to define a filter band that was narrowly graded enough to prevent the possibility of a gap-graded filter from being supplied. The additional provisions in the revised SCS filter design criteria were:

- The width of the design filter should be such that at a given percent finer size between 10 and 60, the coarse side of the designed filter band has particles that are no more than 5 times the finer side of the design filter band.
- The coefficient of uniformity of the filter band should be between 2 and 6. This limits filters from being overly broadly graded and overly narrowly graded as well.

Link_011_Peck (1990)

Peck summarized how a filter zone effectively protects the core of an embankment dam against internal erosion and at the same time serves to reduce flow through any defects by developing a filter cake. The following is from his article published in the H. Bolton Memorial Symposium Proceedings (1990):

> *The writer agrees with Sherard that the available data on performance of dams suggest that the filter adjacent to the core serves a purpose even more vital than has generally been assumed. Undoubtedly, as has long been recognized, the downstream filter limits the amount of material that can be lost from the core by erosion and thereby protects the integrity of the core. In addition, however, it serves as a substitute core where, for any reason, defects in the core have permitted concentrated seepage. Hence, seepage through the dam as a whole may not increase perceptibly*

even when defects develop in the core. The impregnated filter takes over the function of the core.

Link_0012_ICOLD (1994)

The International Committee on Large Dams (ICOLD) published a summary of the status of filter design in 1994 entitled, “Bulletin 95 –Embankment Dams Granular Filters and Drains, Review and Recommendations.” The first caveat in filter design according to this set of recommendations is to determine whether the base soils are broadly graded and potentially internally unstable. The article recommends using methods presented by Kenney and Lau (1985) as well as alternative techniques such as those presented by Sherard (1979) and Lowe (1978). Most of these recommendations can be condensed to an examination of the slope of the particle size distribution curve. Soils with overly flat portions in their grain-size distribution curve may be unstable according to these concepts. Chapuis (1992) also provided valuable insight into this topic.

Link_013_Foster et al. (2001)

This study by Foster et al. (2001) used data from most available previous filter research. The authors re-analyzed the research and concluded that the Sherard criteria were satisfactory for design of filters for new projects, but that alternative criterion could be applicable for existing projects when filters may not meet current criterion. Three boundaries of filter functioning were presented:

1. *No erosion boundary* – Practically no erosion of base soil occurs in preformed 1-millimeter (mm) hole protected by the filter.
2. *Excessive boundary filter* – Filter seals after “some erosion” of base soil. For cohesionless base soils, loss of material from erosion is less than 100 grams for some erosion and greater than 100 grams was termed excessive erosion.
3. *Continuing erosion boundary* – Filter is too coarse to seal in no erosion test – erosion continues in test.

The article presented the following summary of recommendations for design of filters for new dams. A different set of recommendations was presented for evaluating existing dams:

1. Sherard’s criteria should not be relaxed for new dams. The criteria incorporate a significant safety factor.

2. For critical filters designed to protect dispersive clays, No Erosion Filter Tests are recommended. (A critical filter is one that must protect against a crack in the soil through which water is flowing and where erosion of the crack could result in failure of the structure).
3. For Category 1 dispersive clays, rather than using 9 × d_{85}, recommend using 6.4 × d_{85} as more conservative criterion.
4. For Category 2 dispersive clays (< 85% fines), recommend using D_{15} of 0.5 mm rather than 0.7 mm.
5. Percent fines value of 35% should be used to separate Categories 2 and 3 rather than 40%.
6. For Category 4 base soils (< 15% fines), recommend using D_{15} < 7 times d_{85} rather than Sherard criterion of 4 × d_{85}.
7. Recommend different criterion for Category 3 base soils.

Fell and Foster recommendation for Category 3:

$$D_{15} \leq 1.6 \times \left[\frac{(35 - A)}{20} \times (4 \times d_{85} - 0.7) \right] + 0.7$$

Link_014_Milligan (2003)

Milligan, in his 2003 Terzaghi lecture (Milligan 2003), provided several recommendations regarding filter design. Some of the recommendations are as follows:

- The use of single, widely graded one-stage filters should be avoided; multistage, narrowly graded filters are recommended.
- A uniform sand, with a maximum size not exceeding 30 millimeters (mm) and a D_{15} size less than 0.7 mm, is recommended as the preferable downstream filter for most core materials provided the filter is wetted during placement to inhibit segregation.

Link_015_Comparison of U.S. Gov't Agency Criteria

The filter gradation research published by Terzaghi (1922), expanded upon by Bertram (1940), and detailed by Sherard and Dunnigan (1985) forms the basis for filter design (grain size gradation criteria) for that is currently used by the U.S. Army Corps of Engineers (USACE), Bureau of Reclamation (Reclamation), and the Natural Resources Conservation Service (NRCS, formerly the Soil Conservation Service) (Navin et al. 2006). Each of these agencies has their own design procedures listed in individual guidance documents included in the list of references. The criteria of the three agencies are very similar, with minor variations (Talbot and Pabst 2006).

Natural Resource Conservation Service (NRCS)

The U.S. Department of Agriculture's Soil Conservation Service (SCS) is now known as the NRCS. The NRCS filter design criteria is based on the results of an extensive laboratory filter study carried out by the SCS at their Soil Mechanics Laboratory in Lincoln, Nebraska, from 1980 to 1985. The principals involved in this study were Lorn P. Dunnigan, SCS (retired), James R. Talbot, SCS (retired), and James L. Sherard, consultant (deceased). Dr. Sherard's SCS effort arose from an earlier joint study regarding dispersive clay soils in dams. His work with SCS during the filter research studies was performed under a cooperative agreement in which he devoted his personal time and expense to ensure its success (McCook and Talbot 1994).

Results of the 1980 to 1985 research were published in Sherard et al. (1984), Sherard and Talbot (1984), and Sherard and Dunnigan (1989). In 1986, the SCS published a revised Soil Mechanics Note No. 1, "Guide for Determining the Gradation of Sand and Gravel Filters," containing criteria from the research effort (U.S. Department of Agriculture 1986). The criteria and examples in the revision resulted in poor feedback ratings from field engineers due to poorly defined gradation limits and construction control measures. Additional criteria to restrict the gradation limits were developed by SCS to provide more uniformly-graded filter material and to prevent gap-graded or skip-graded filters from being designed.

The revised Note No. 1 was developed in 1993 by SCS engineers Danny K. McCook and Charles H. McElroy of the Soil Mechanics Laboratory in Fort Worth, Texas, and James R. Talbot (retired SCS). The revisions are found in NRCS' latest guidance, issued in October 1994, as Part 633 National Engineering Handbook, Chapter 26, Gradation Design of Sand and Gravel Filters (U.S. Department of Agriculture 1994).

U.S. Army Corps of Engineers (USACE)

Filter gradation design guidance has evolved to include more details illuminated by research primarily accomplished during the 1980 to 1985 SCS efforts. The three major design manuals for embankments, Engineering Manual (EM) 1110-2-2300 (Dams), EM 1110-2-1901 (Dam Seepage), and EM 1110-2-1913 (Levees) include appendices for filter gradation design.

Historically, the Corps' filter criteria included two equations for erosion stability (particle retention) within the filter, one equation for filter permeability, and two equations for pipe conduit filter design ("D" is filter gradation size, and "d" is base (protected soil) gradation size):

- $D_{15} < 5d_{85}$
- $D_{50} < 25d_{50}$
- $D_{15} > 5d_{15}$
- D_{50} > hole diameter, for circular opening in perforated pipes and screens
- $D_{50} > 1.2 \times$ slot width, for slotted opening in perforated pipes and screens

The 1978 version of EM 1110-2-1913 included the above equations, as did the 1986 version of EM 1110-2-1901. The 1993 version of EM 1110-2-1901 and the 1994 version of EM 1110-2-2300 were significantly modified to incorporate the SCS design steps, and included Perry (1987). The criteria were based on the SCS Note No. 1 (U.S. Department of Agriculture 1986) prior to its 1994 revision.

Current USACE filter design guidance is found in Appendix B (Filter Design) of EM 1110-2-2300 (30 July 2004), Appendix D (Filter and Drain Design and Construction) of EM 1110-2-1901 (30 April 1993), and Appendix D (Filter Design) of EM 1110-2-1913 (30 April 2000). The guidance in all three documents is the same as that in the previously referenced 1986 SCS Note No. 1. USACE is updating filter design guidance (Appendix D, EM 1110-2-1901) to include the 1993 revisions to the SCS Note No. 1, referenced in the Part 633 National Engineering Handbook, Chapter 26, Gradation Design of Sand and Gravel Filters (U.S. Department of Agriculture 1994).

U.S. Department of Interior, Bureau of Reclamation (USBR)

Similar to USACE efforts, the USBR filter gradation design guidance has evolved to include more details illuminated by research primarily accomplished during the 1980 to 1993 SCS (NRCS) efforts. The major design manual for embankments, Design Standards No. 13, Chapter 5 (U.S. Department of the Interior 2007) is devoted to protective filter gradation design.

The USBR Design of Small Dams manual (U.S. Department of the Interior 1987) allowed the following filter criteria ("D" is filter gradation size, and "d" is base (protected soil) gradation size):

- $D_{15} < 5d_{85}$
- $D_{15} > 5d_{15}$
- $D_{85} > 2 \times$ maximum opening in pipe drain

A more complete discussion of filter criteria was given in older versions of the USBR Design Standards No. 13 (Embankment Dams), Chapter 5 (Protective Filters) but the above criteria formed the essential framework for filter gradation design.

The latest USBR filter guidance (20 March 2007 update to the Design Standards No. 13, Chapter 5) completely replaced the previous version (27 August 2004). The following topics were updated:

- broadly graded soils and internal instability
- filter arrangement with emphasis on single versus two-stage drains
- construction considerations including handling, placing, and compaction
- pipe slot size criteria
- minimum dimensions of chimney filters and blankets

Additions to the standard included:

- caution on design of filters finer than portions of the foundation material (filter barriers)
- the concept of filter classes
- filter requirements based on their intended purpose

- commercially available materials that can be used for typical filter and drain applications.

Comparisons and recommendations

Comparisons, refinements, and recommended modifications to existing criteria as implemented in this manual are summarized as follows:

- Several of the agencies' current criteria require base soils with gravel particles to be computationally re-graded from the No. 4 sieve size before designing the filter gradation. Reclamation's procedure does not require re-grading on the No. 4 sieve if the base soil has certain characteristics. Those characteristics are that:
  - The sample has less than 15% finer than the No. 200 sieve
  - The base soil is not gap-graded
  - The base soil is broadly graded according to definitions used in the Unified Soil Classification System
- The USACE procedure suggests re-grading from the No. 30 sieve size if the base soil is gap graded (see Figure B-1 in USACE EM 1110-2-1913).
- The recommended procedure in this manual is to re-grade the base soil from the No. 4 sieve size in all cases unless the base soil has less than 15% fines and is not gap-graded.
- Gap-graded soils should be re-graded to design a filter that will protect against the finer fraction of the sample.
- Current filter criteria use a table of allowable D_{90} sizes that depend on the minimum D_{10} of the filter being designed. By limiting the large sizes in a filter, segregation is prevented. A supplemental tool is available based on research by Sutherland and Grabinsky (2003) for additional evaluation of segregation potential of filters.
- Foster and Fell (2001) recommended a slightly more conservative filter criterion (a finer filter) for Category 1 and 2 soils with dispersive clay fines. This is a worthwhile addition to criteria and was incorporated into the procedures recommended in this manual for both Category 1 and 2 soils.
- The boundaries defining the four base soil categories are the same for the three Federal design agencies and are the same as those recommended in this manual.

Filter bandwidth

The discussion below provides a background on limiting the width of the designed filter band. The intent of this requirement is to not allow design of a gap-graded or overly broadly graded filter. The guidance is based on the NRCS criterion. Both the NRCS and the USACE incorporate these requirements into their criteria.

The two requirements to prevent these problems in a design filter are:

- The coefficient of uniformity (equal to $D_{60} \div D_{10}$) of the upper and lower filter limit boundaries should be between 2 and 6.
- The width of the design band should be such that at any given percent finer between 10 and 60 percent, the maximum diameter should be not more than five times the minimum diameter.

Reclamation's approach to achieving the goal of having a filter band that is not overly broadly graded is to require that the gradation limits near the middle of the curve should not differ more than 35 points (35%) from one side of the design band to the other. This results in a much narrower band than the NRCS/USACE criteria. Using the Reclamation criterion for band width may result in a more costly product if suppliers understand the implications of supplying a filter that will plot within a narrower band. The width of the band required by the Reclamation criterion varies with the steepness of the band according the following relationship:

Coefficient of Uniformity of Band	Width of Band to Achieve Δy of 35%
6	3.5
4	2.6
2	1.6

The recommendations in this manual follow the NRCS/USACE criterion for the width of the filter band.

Maximum filter particle size

The current criteria of the NRCS is a maximum particle size of 3 in., as is the criterion of the USACE. A planned revision to the NRCS criteria will reduce the maximum particle size in a filter to 2 in. Reclamation already requires a maximum particle size of 2 in., and the recommended procedure in this manual also requires a maximum particle size of 2 in.

Filter particle segregation

All three agencies use the same criterion to prevent segregation of filter particles. The criterion is represented by a table that specifies an allowable ratio of the D_{90} to the D_{10} sizes in the designed filter. Based on research by Sutherland and Grabinsky (2003), a supplemental evaluation of segregation can be performed for coarse filters by computing a segregation index. This is an optional evaluation that is seldom required and only then for coarser filters.

Filter permeability

The agencies' criterion for permeability is that the filter band has a minimum D_{15} size that is a multiplier of the d_{15} size of the base soil (before re-grading). The NRCS criterion is a multiplier of 4. The USACE requires a multiplier of 3 to 5, and Reclamation requires a ratio of 5. The recommendations in this manual are to design the filter with the paramount goal of meeting the filter criterion and then using the highest possible value of D_{15}, which is based on an acceptable minimum width in the filter design band.

In some filter designs, this may cause the design filter band to have a smaller D_{15} size than is desirable from a permeability standpoint. In those cases, the recommendation is that the filtering requirement is always given precedence, and the minimum D_{15} is then controlled only by the allowable width of the band. Permeability of the filter is a function of the square of the D_{15} size. For example, if the designed filter has a D_{15} that is three times the D_{15} of the base soil, the permeability of the filter will be nine times that of the base soil. The designed filter should have a D_{15} that is as large as possible without limiting the width of the design band to a width that is so narrow it would be impractical to economically process and furnish on a consistent basis.

Filter fines

The three agencies' criterion for percent finer than the No. 200 sieve are similar. The basic requirement is that filters should have less than 5% finer than the No. 200 sieve. Reclamation adds a requirement that filter materials in the stockpile (i.e., before being compacted) must have 3% or less fines because some breakdown of the filter during construction is assumed, which will increase the percentage of fines.

Link_016_USACE Base Soil Selection

U.S. Army Corps of Engineer guidance on plotting gradation curves to represent the base soil for which the filter is being designed is as follows:

> *Collect sufficient samples to understand the composition of the base soil. Use enough representative sample gradation tests to define the expected range of grain size curves for the base soil or soils. When evaluating a base soil, obtain numerous representative sample grain size distributions and plot them all on the same gradation curve sheet, so that the normal range or band of soil gradations can be seen along with any outlier gradations. A decision must be made whether or not to design a filter/drain for the normal range or all soil gradations including the outlier gradations.*
>
> *Where base soils (e.g., impervious core material) will be selectively borrowed and mixed and blended, it may be reasonable to design for the typical gradation band and exclude outlier gradations. Where the base soil will not be altered by construction (e.g., in-situ foundation materials), outlier gradations must be carefully evaluated, and if necessary, a special filter designed and constructed.*

Per Appendix B (Filter Design), EM 1110-2-2300, dated 30 Jul 04:

> *Determine the gradation curve (grain-size distribution) of the base soil material. Use enough samples to define the range of grain size for the base soil or soils and design the filter gradation based on the base soil that requires the smallest D_{15} size.*

Link_017_Reclamation Base Soil Selection

Bureau of Reclamation guidance on plotting gradation curves to represent the base soil for which the filter is being designed is summarized as follows:

> *It is desirable to plot each gradation curve from all samples from a specific borrow source on the same sheet. For example, plot on one sheet all gradation curves of material that are to be used for an impervious zone. The normal range of the impervious material, as well as outlier gradations (gradations that do not fit within the normal range), can then be seen.*
>
> *Design the filter using the base soil that requires the smallest D_{15} size for filtering purposes.*

Link_018_Prevent Gap-Grading

To prevent gap-graded filters
Both sides of the design filter band will have a coefficient of uniformity defined as CU = $D_{60} \div D_{10}$, equal to or less than 6. Initial design filter bands by this step will have CU values of 6. For final design, filter bands may be adjusted to a steeper configuration, with CU values less than 6, if needed. This is acceptable so long as other filter and permeability criteria are satisfied. Filters should not be designed with a CU value less than a value of 2, as this would be a very poorly graded filter that could be difficult to obtain and would not easily compact. Initial bands are often steepened to accommodate the use of a standard commercially available gradation that would require a more uniform gradation.

Additional design considerations

Note that step 7 of the procedures provides for a filter band design that is as widely, or well-graded, as possible. This usually provides the most desirable filter characteristics. However, in some cases, a more uniform gradation may be desired such as when a standard commercial gradation is available or when it is desirable to use onsite materials that are more uniformly graded. In these cases, the filter limits that define the band can be steepened to accommodate the more uniform graded material. The limits can be steepened such that the CU is less than 6, but no less than 2. In no case can the filter band be made more flat (CU greater than 6) to accommodate a more well-graded or widely graded material. In making the limits steeper, only the top portion of the filter band can be moved. The limits set for the D_{15} must remain as designed in step 5 to meet the filtering criteria.

The requirements for coefficient of uniformity apply only to the coarse and fine limits of the design filter band. It is possible that an individual, acceptable filter whose gradation plots completely within the specified limits could have a CU greater than 6 and still be acceptable. The design steps of this procedure will prevent use of gap-graded filters. It is not necessary to closely examine the coefficient of uniformity of a particular filter so long as it plots within the design filter band.

Link_019_Segregation

To ensure that the filter cannot easily segregate during construction, the filter must not be overly broad in gradation. The relationship between the maximum D_{90} and the minimum D_{10} of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. (This factor of 1.2 is based on the assumption that the slope of the line connecting D_{15} and D_{10} should be on a coefficient of uniformity of about 6.) Then determine maximum D_{90} using Table 5-4.

Sand filters with a D_{90} less than about 20 millimeters (mm) generally do not require special adjustments for the broadness of the filter band. For coarser filters and gravel zones that serve both as filters and drains, the ratio of $D_{90} \div D_{10}$ should decrease rapidly with increasing D_{10} sizes.

Sutherland and Grabinsky (2003) recommend computing a size modulus, S_m, to evaluate whether a filter may be overly broad in gradation and subject to segregation. Occasionally, this index should be evaluated when filters are proposed with D_{90} sizes larger than 20 mm. The method of computing S_m is as follows:

- Plot the filter band on a gradation curve and determine the D_{max} and minimum D_{30} sizes for the two sides of the designed filter band. D_{max} is obtained from the coarse side of the filter band being designed, and D_{30} is obtained from the fine side of the band. The following guidelines should be used to determine if the filter being designed is overly broad. The filter design should be designed to be more narrowly graded if the S_m value is < 0.25.

$$S_m = \frac{\log \frac{4.75}{D_{30}}}{\log \frac{D_{max}}{D_{30}}}$$

Link_020_Historical Background of Lab Studies

Sherard and other researchers conducted experiments in the 1980s that examined the mechanisms by which filters could protect against internal erosion of a defect in a base soil. Most previous filter research had examined intergranular flow through sands. Sherard's interest in hydraulic fracturing, his experience with sinkholes occurring in embankments with broadly graded soils, and the failure of numerous small embankments constructed of dispersive clay all provided the impetus for his studies. The Soil Conservation Service (SCS) (now the Natural Resources Conservation Service) solicited Sherard's assistance to evaluate the failures of some of their embankments, and he used the SCS laboratory in Lincoln, Nebraska, to conduct experiments to investigate the problem of flow through defects in embankments

Link_021_Rationale for Inclusion of Crack in No Erosion Filter Test

The earlier paper on filters for clays and silts presented two different laboratory tests, the slot test and the slurry test, which gave the same results (Sherard et al. 1984b). For any given silt or clay, these tests could be used to define a filter boundary size, $D_{15}B$, separating successful and unsuccessful tests. For both the slot and the slurry tests, there was a small amount of erosion of the base specimen into the successful filter during the test. A filter was judged to be successful when the flow rate rapidly decreased and stabilized with a small constant flow of clear water.

Later in the research program, another test was adopted in which it was possible to define a filter boundary size, $D_{15}b$, at which "no visible erosion" of the walls of the preformed leakage channel took place during the test. Tests with filters slightly coarser than the boundary had visible erosion. This test, the no erosion filter (NEF) test, was also found to work *very well for coarse-grained impervious soils, whereas the slot and slurry tests were found to be not satisfactory for impervious soils with D_{15} size much greater than about 0.1 mm. The details of the laboratory test setup are shown in Attachment F.*

The filters were made as described for the slot and slurry test (Sherard et al. 1984b) by blending together measured weights of clean sand of various sizes to precisely control gradation and the D_{15} size of the filter. The soil was compacted in place on top of the filter (Karpoff 1955; Lund 1949; Sherard et al. 1984b) at a water content near standard Proctor optimum. The water content and density of compacted base specimens were not measured routinely. The research showed that the test results were not influenced significantly by moderate differences in water content or compacted density. The fine-grained soils and the coarser impervious soils with plastic fines were cured at a water content satisfactory for compaction for at least 24 hr before compacting them in the test cylinder.

Link_022_Material Types

Three methods for grouping aggregates have been used. One is related to the general composition of the aggregates—specifically, what type of rocks form the parent material for the aggregates and what minerals are in the rock. The second grouping of aggregates is according to the source of the aggregate. The third category of description for aggregates relates to the origin of the rocks forming the aggregate.

A discussion of each category of aggregate description follows:

- General Composition
    - *Mineral*: Naturally occurring substance with an orderly structure and defined chemistry.
    - *Rock*: Mixture of one or more minerals.
- Source
    - *Natural sands and gravels*: Formed in riverbeds or seabeds and usually dug from a pit, river, lake, or seabed; sands are fine aggregates and gravels are coarser particles.

- ○ *Manufactured aggregate (crushed stone or sand)*: Quarried in large sizes and then crushed and sieved to the required grading – also crushed boulders, cobbles, or gravel.
- ○ *Recycled*: Made from crushed concrete (not used for drains and filters).
- Origin
  - ○ *Igneous*: Cooled molten material; includes siliceous materials primarily consisting of compounds of silica (e.g., granite).
  - ○ *Sedimentary*: Deposits squeezed into layered solids – includes carbonate materials from deposited sea shells (e.g., limestone).
  - ○ *Metamorphic*: Igneous or sedimentary rocks that have been transformed under heat and pressure.

Link_023_Basic Tests

Gradation

Perhaps the most important property specified for filters is a specified range of acceptable gradation of the aggregate. Filters are usually specified in terms of an acceptable range of gradation that includes a minimum and maximum percent finer than a given range of sieve sizes.

The gradation of a filter source being evaluated is measured by two basic tests:

- ASTM C136, Sieve Analysis of Fine and Coarse Aggregates, determines the overall gradation of the sample
- ASTM C117, Materials Finer than 0.075-mm (No. 200) Sieve in Mineral Aggregate by Washing

Gradation testing uses an additive to the water used for testing to ensure complete dispersal of any clay lumps in the aggregate.

Figure L-23-1 illustrates how gradation tests are used to determine whether a supplied filter meets gradation requirements. The red gradation curves represent the required gradation for the filter. The green curve is the plotted gradation resulting from ASTM tests on a proposed filter. In this example, the supplied filter does not plot within the required gradation band and would not be acceptable.

Specific gravity

While specific gravity is not often part of specifications for filters, a brief discussion is included in this manual for general information purposes. The tests are more commonly used in working with aggregates for concrete production. Several types of specific gravity measurements relate to the type of test performed. Test procedures are different for coarse aggregates and fine aggregates. ASTM C127 is used for coarse aggregate (larger than the No. 4 sieve), and ASTM C128 is used for measuring specific gravity of fine aggregates (passing the No. 4 sieve). During the test, a value for percent absorption is obtained, which reflects the amount of water a dry aggregate will absorb. Some specifications, particularly those for aggregates to be used in concrete production, limit% absorption to a maximum value because aggregates with high absorption usually are poorer quality and have a particularly low resistance to freeze-thaw.

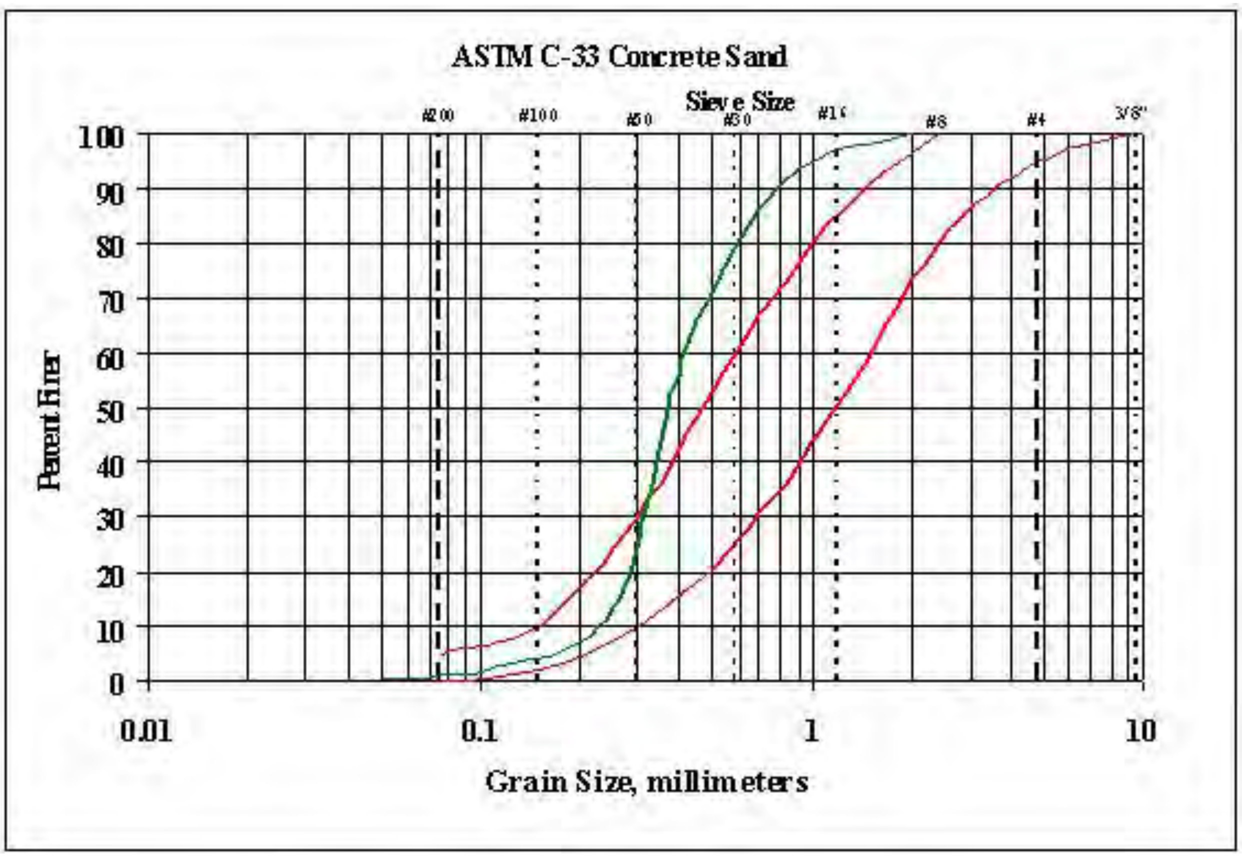

Figure L-23-1—Specification for ASTM C-33 concrete sand (red) and supplied filter that has a gradation not meeting the specification (green). Specifications require a supplied filter to have a gradation that plots within a required range of gradations, the design band. Note: customary X-axis scale is reversed.

Link_024_Problems with the Vaughan Test

The procedure outlined by Vaughan was originally intended to identify filters that have an excessive amount of clay fines that may cause the filter to have unacceptable cohesive behavior. The test has never been standardized to any extent to permit a procedure that is repeatable by laboratories. The items in the test that are not fully detailed and could significantly influence the test results are:

1. What is the energy used in compacting the sand into a standard Proctor mold? Presumably, ASTM D698 energy would be used, where three lifts of soil are evenly compacted into the mold using 25 blows of a 5.5-lb hammer dropped 12 in. per blow.
2. What is the water content used to mold the sample? Samples molded at less than about 4% water content presumably would have little ability to retain their shape when extruded from the mold.
3. Are the samples to be placed immediately in a water pan or should the sample be allowed to air dry before placing it in the pan of water?
4. Should water be added to the pan or should the sample be submersed in a water bath using a cradle of some fashion?
5. At what point during the submergence of the sample should the sample be judged as to its behavior—when the sample is 50% submersed, fully submersed, or at some other intermediate submergence level?

A number of issues have been identified in the Vaughan Test and research was undertaken by Virginia Polytechnic Institute (VPI) to evaluate the Vaughan Test further. Testing was intended to determine the usefulness and to devise a modified test with a more standardized approach. The procedure proposed by the VPI research is summarized as follows:

1. The sample of filter is compacted into a steel mold that is 8 in. high and has a 4-in. diameter. The mold has a 2.5-in. collar to permit slight overfill of the mold followed by trimming. The mold is constructed as a split mold to allow removing the mold from the sides of the compacted specimen with a minimum of disturbance.
2. The sample is compacted using the modified Proctor (10-lb hammer dropped 18 in. each blow) hammer, using 44 blows per layer and using five layers to fill the 8-in. mold. The increase in number of blows per layer (25 blows per lift are used in the modified Proctor test) was done to ensure that the modified compaction energy was achieved. The energy applied is

equal to 56,720 ft-lb per cubic foot compared to the 56,250 nominal effort of the ASTM D1557 test.

3. After compaction, the split mold is removed and the sample is placed in a container (a pan) with sides at least 8 in. high. The container should have a smooth level bottom to support the sample evenly.
4. The slump (if any) of the sample in an as-molded condition is measured by carefully placing a straight edge on the top of the mold and gently pressing into the mold, measuring any deformation vertically. If the sample collapses under this slight amount of pressure, that is noted as excellent self-healing behavior.
5. The sample is submersed as quickly as possible and the slumping of the sample is measured and expressed as in. of vertical slump. Readings are taken periodically beginning with the submergence of the sample.

To date, not enough testing has been performed to allow quantitative guidelines to be established.

Link_025_Functions

A zone of granular material of designed gradation may serve a single function, or it may serve multiple functions, depending on the design. The primary functions of granular zones are:

- *Filtration* – A filter blocks particles from a base soil by flow that is carrying eroded particles caused by either intergranular seepage or preferential flow. Preferential flow is that occurring through a crack within the soil itself or at the contact between a soil and an appurtenance or bedrock foundation. In the case of a base soil without defects, the filter is designed to prevent backward erosion piping. In the case of a base soil with potential or existing preferential flow paths, the filter is designed to prevent incipient or continuing internal erosion. In either case, a properly designed filter will allow water to pass through the filter and at the same time will block movement of soil particles. Chapter 1 has more detail on filter theory.
- *Drainage* – A drainage zone provides relief of intergranular pressures by conveying all the seepage flow arriving at the interface between the drain zone and the base soil or bedrock carrying the seepage flow. Many granular zones associated with concrete structures have a primary function of providing relief of uplift pressures. Any drainage zone

must also be designed to be filter compatible with the base soil and any collector system, including slotted or perforated drainpipes.

- *Transition (same as filter)* – A zone or zones of progressively coarser granular material that provides transition in gradation between zones of an embankment. In essence, this is the same function as the filter function.

Link_026_Limitations of Dual Function

A granular zone that is designed to collect large flow quantities must have a relatively coarse gradation to have adequate permeability to accept the large flow quantity. A basic tenet of permeability is that the coefficient of permeability is a function of the effective grain size of the granular material. If large quantities of flow must be intercepted and conveyed by the granular zone collecting the flow, the D_{10} size of the zone must be relatively large. At the same time, the ability to filter particles from the base soil that the drain contacts is a function of the D_{15} size of the filter. Having a large D10 size to achieve high permeability then is self-defeating for providing a filtering capability to the zone.

Dr. Karl Terzaghi's (Terzaghi 1925) original examination of the functioning of a filter discussed this dual function well:

> *When flowing under head through permeable ground, the water exerts on the soil a pressure. In every point of the underground the pressure acts in the direction of the flow, and its intensity is directly proportional to the local hydraulic gradient. . .The only efficient remedy is this: to keep the soil in the danger zone down, by means of a graded filter. The filter safeguards the free escape of the seepage water, and at the same time it prevents the soil particles from drifting away.*

An example of a situation in which a filter zone is called upon to serve a dual function is a coarse blanket drainage zone placed on a foundation. If the foundation on which the blanket drain lies is conveying large flow quantities either through solution features or large open joints, the blanket drain must not limit this flow with a fine filter because the flow will merely find a path around the filter. At the same time, if the flow is carrying fines eroded from the embankment in contact with the rock, a finer filter is required to block the flow of the soil fines. In a situation like this, the filter/drainage zone has a dual function that cannot be satisfied. The only tenable solution to this problem is to limit the flow quantity by some type of cutoff of the seepage, whether it be a grout curtain, a rolled fill cutoff trench, or some other means.

Figures L-26-1 through L-26-3 show a dam where a concentrated large amount of seepage was exiting the ground at the groin in the abutment. Seepage was being carried by jointed rock with significant openings, and a filter with sufficient capacity to collect this quantity of seepage would necessarily be relatively coarse gravel. A gravel that would be coarse enough to convey this amount of seepage could not be an effective filter for any soil being eroded from the embankment in contact with the bedrock. Fortunately in this case, collected seepage water contained no fine-grained particles.

Figure L-26-1. Water discharging in the groin area downstream of an embankment during higher than normal reservoir storage. Seepage was through bedrock in the foundation that was not grouted. A gravel drainage layer was placed over the seepage to provide a controlled outlet. The seepage was not carrying fines, which would have been problematic to deal with because a fine filter would be required to block the fines but a coarse filter was required for capacity. In this case, the gravel layer provided a collector mechanism, with no filtering being required.

Figure L-26-2. Gravel drain being placed over seepage area.

Lowering the reservoir as quickly as possible is needed to reduce the head and velocity of the flow, and remedial action to seal the bedrock openings perhaps accompanied by a filter/drain to collect minor amounts of flow bypassing the remedial cutoff would be the best approach. This dam was treated using these methods and likely will not experience similar problems in the future. More detail on this site is included in Attachment E.

Figure L-26-3.—Although a gravel filter was placed over the flow area, it was so coarse that it would not have been an effective filter for eroding fine particles, had there been any. Fortunately, in this situation, the flow in the bedrock was not in contact with the embankment, and no erosion of the fill occurred.

Link_027_Design to Satisfy Function

Ordinarily, a single set of criteria is used for designing a filter to be compatible with the base soil with which it is in contact. In other words, different criteria are not normally applicable for a zone with a single function as a filter, or a single function as a drain, or a dual function. A single universal set of criteria is ordinarily used regardless of the function. The reason for this is that usually a zone designed to be an effective filter will have an adequate permeability to serve as a drain because of the relationship between the D_{85} and the D_{15} of the base soil. Using criteria as shown in this manual, the maximum allowable D_{15} of the designed filter is based on the d_{85} of the base soil. In the example shown in Figure L-27-1, the base soil is a reasonably well-graded silty sand with fines that are not dispersive. It falls in Category 3 because it has 19% finer than the No. 200 sieve. Using criteria for Category 3 soils, the maximum allowable D_{15} size is computed to be 1.8 millimeters (mm).

$$\leq \left[\frac{\mathbf{40-A}}{\mathbf{40-15}}\right]\left[\left(\mathbf{4^{*}\,d_{85}}\right)-\mathbf{0.7mm^{*}}\right]+\mathbf{0.7mm^{*}}$$

$$\leq \left[\frac{\mathbf{40-19}}{\mathbf{40-15}}\right]\left[\left(\mathbf{4^{*}\,0.5}\right)-\mathbf{0.7mm^{*}}\right]+\mathbf{0.7mm^{*}}=\mathbf{1.8\,mm}$$

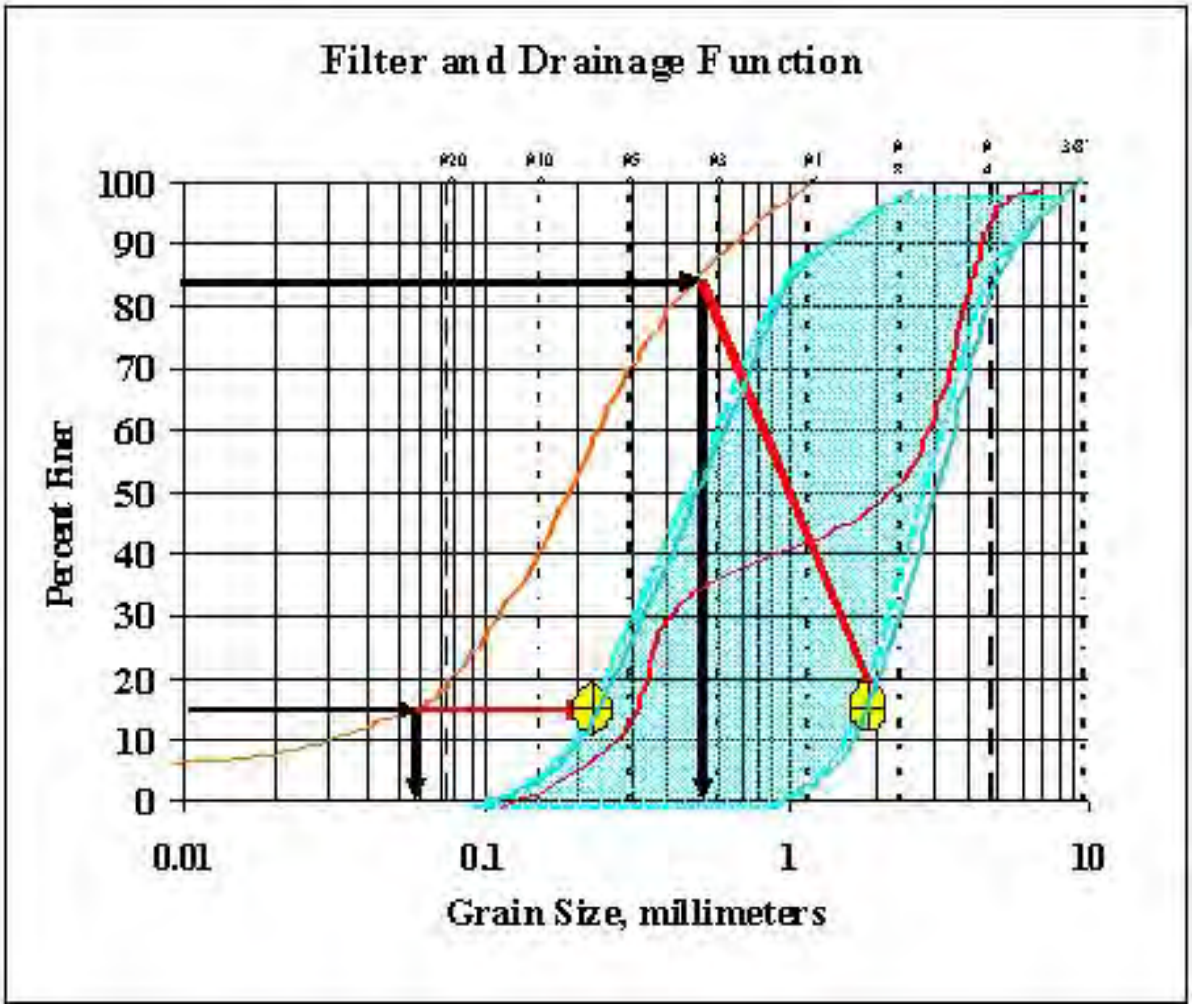

Figure L-27-1.—Illustration of range of filter/drain gradations that concurrently satisfy filter and permeability requirements. Base soil is reasonably well-graded silty sand. Note X-axis scale reversal from convention.

The minimum desirable permeability of a filter is achieved by having a filter with a D_{15} size that is at least four times the d_{15} of the base soil. In this example, the base soil has a d_{15} size of 0.058 mm, so the filter should have a D_{15} size at least equal to 4×0.058 mm = 0.23 mm. A filter that would meet only these two basic criteria could be quite wide, as shown in Figure L-27-2.

A filter would not be designed with this wide a band because it would allow gap-graded gradations to be furnished. As discussed in Chapter 5, the widths of filter band designs are usually restricted to prevent the possibility of a gap-graded filter being supplied. In Figure L-27-1, the purple line shows an undesirable gap-graded filter that would meet the requirements

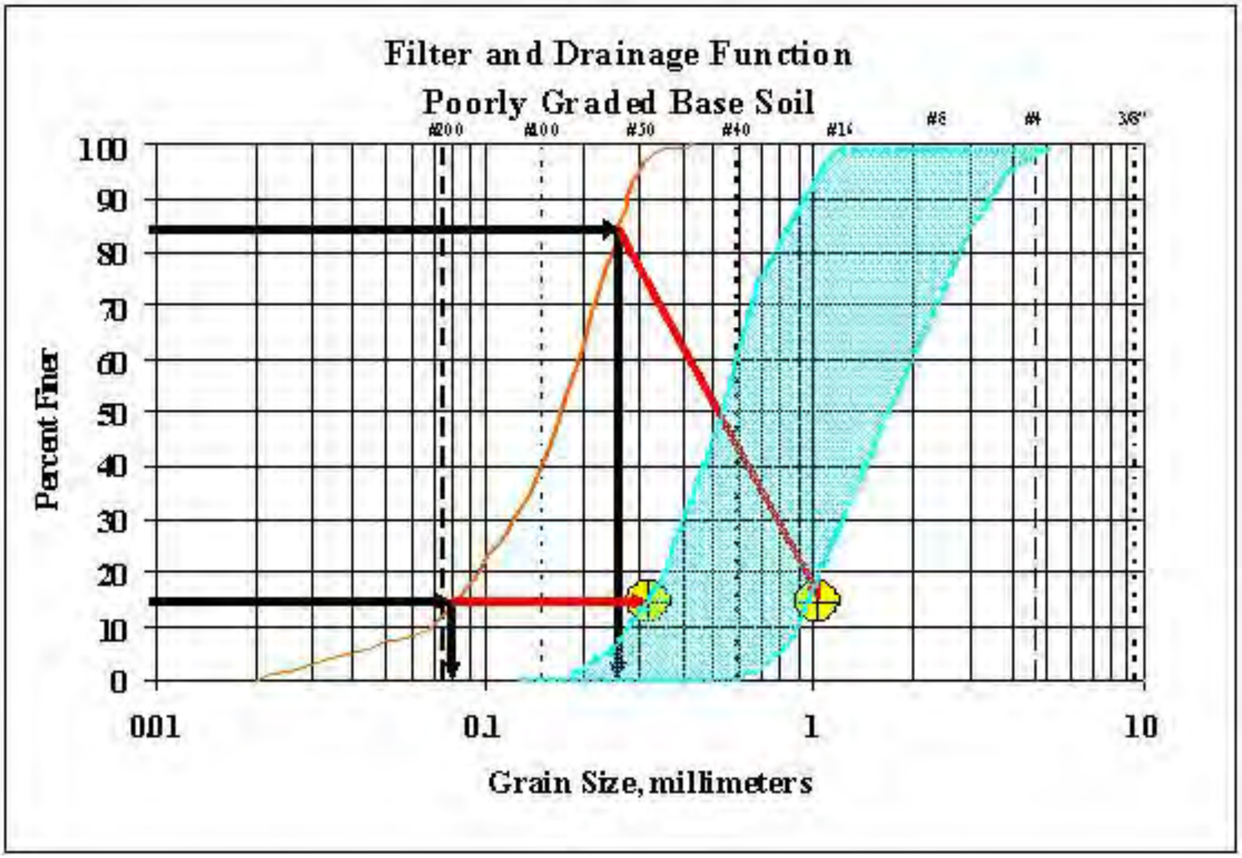

Figure L-27-2. Illustration of range of filter/drain gradations that concurrently satisfy filter and permeability requirements. Base soil is poorly graded silty sand.

of the specified band, but would be undesirable in its properties because it is gap-graded.

The purpose of Figure L-27-1 is to demonstrate that ordinarily meeting both filter and permeability criterion is easily accomplished for a well-graded base soil. If a base soil were more poorly graded, a narrower range of gradation of filter/drain material would be required to satisfy both the filter and drainage requirements.

Figure L-27-2 illustrates this point. The filter band that is fine enough to meet criteria for filtering and coarse enough to meet permeability requirements is considerably narrower than the possible band shown in Figure L-27-1. In this example, the base soil is in Category 4 using procedures of Chapter 5. The maximum allowable D_{15} size of a filter for a Category 4 base soil is equal to four times the base soil d_{85} size.

In this example, the base soil can be seen to have a d_{85} size of about 0.26 mm, so the maximum allowable D_{15} size for the filter is 4 × 0.26 = 1.04 mm. A filter with desirable permeability will have a D_{15} size that at least four times the d_{15} size of the base soil. The base soil has a d_{15} size of about 0.08 mm, so the filter should have a D_{15} size no smaller than 4 × 0.08 = 0.32 mm. A filter band that meets both filter and permeability requirements then will have a D_{15} size of between 0.32 mm and 1.04 mm. This represents a design band that has a ratio of maximum to minimum particles sizes of about 3.3. This narrow a band might be slightly more difficult to meet than a wider band, but is not impractical. For comparison, the width of the frequently specified ASTM C-33 fine concrete sand is about 2.5.

Link_028_References for Filter/Drainage Zones in Embankments

Some of the following documents with recommendations for filter and drainage design were written in a period before advances were made in the understanding of preferential flow paths and internal erosion in embankments and foundations. Some of these historical documents primarily discuss intergranular seepage forces and backward erosion piping without discussing internal erosion and preferential flow. Readers should understand that preferential flow through cracks in the dam or core section may be as important, or more important, than intergranular seepage when designing filter and drainage zones. Modern embankment design emphasizes the use of chimney filter zones to control not only intergranular seepage but to intercept hydraulic fractures in the embankment and prevent internal erosion failures. This facet of design is covered in more detail in Chapter 2.

The primary publications pertinent to design of filter/drainage zones from major embankment design agencies are as follows:

U.S. Army Corps of Engineers – Engineering Manual EM 1110-2-1901, Seepage Analysis and Control for Dams. September 1986.

U.S. Army Corps of Engineers – Engineering Manual EM 1110-2-2300, General Design and Construction Considerations for Earth and Rock Fill Dams, Chapter 6, Seepage Control.

Much of the discussion in this manual is similar to that in EM 1110 2 1901. Chapter 6 includes discussions on seepage control in the embankment and in the foundation.

Three basic methods for seepage control are listed for embankments and include:

- Using flat side slopes that are resistant to sloughing from seepage forces without drains
- Zoning the embankment using a core of less permeable soil and shells of more permeable soil to control the phreatic line and reduce problems with drawdown
- Vertical (or inclined) embankment drains and horizontal drains

The methods listed in the chapter for control of seepage in the foundation are:

- Horizontal drains
- Cutoff measures, including compacted backfill trenches, slurry walls, and concrete walls
- Upstream impervious blankets
- Downstream seepage berm
- Relief wells
- Trench drains
- Drainage galleries
- Grouting of rock foundations

Chapter 6 includes numerous examples of embankment designs in which these methods of seepage control are schematically illustrated in typical embankment cross sections.

USACE ER 1110-2-1806 (July 95).

This document includes considerations for filter/drainage zones used in designs of embankments in seismic areas. Some of the recommendations pertaining to embankment filters are:

- Wider transition and filter sections as a defense against cracking
- Use of rounded or subrounded gravel and sand as filter material

- Near-vertical drainage zones in the central portion of the embankment
- Well-graded core and filter materials to ensure self healing in the event cracking should occur

Bureau of Reclamation. Design Standards, Number 13, Embankment Dams, Chapter 8, Seepage Analysis and Control. 1987.

This document includes methods of analyzing seepage and safety factors that are not included in this manual. Methods for controlling seepage that include filter and/or drainage zones that are discussed in the manual include:

- Embankment zoning (core and shells)
- Chimney drains
- Drainage blankets (blanket drain)
- Toe drains
- Downstream drainage trenches
- Relief wells
- Downstream seepage berms
- Drainage tunnels
- Semihorizontal drain borings

At the end of this chapter is a figure taken from the manual that illustrates the major seepage control features used in an embankment design.

Bureau of Reclamation. Design of Small Dams. 1987.

This document includes sections on seepage control by cutoffs and filter/ drainage zones. It is a standard reference used for many years by earthen embankment designers.

Natural Resource Conservation Service. Soil Mechanics Note SM-3. Soil Mechanics Considerations for Embankment Drains. May 1971.

This manual lists the following categories of drains:

- Vertical and inclined embankment drains
- Horizontal blanket drains
- Foundation trench drains

- Relief wells

The document contains numerous design examples on capacity of drains and pipes in drains. It also has a shortcut procedure for designing small relief wells.

Natural Resource Conservation Service. National Engineering Handbook, Section 628, Chapter 45, Filter Diaphragms.

This document contains detailed guidance from the Natural Resource Conservation Service on design of filter diaphragms around conduits.

Link_029_Use of Chimneys in Embankments

Several prominent experts have expressed opinions on the need for chimney filters in embankment design as follows:

J.L. Sherard (1984) provided the following advice and conclusions on chimney filters:

> *I believe there is already sufficient evidence from dam behavior, supported by theory, to require the designer to assume that small concentrated leaks can develop through the impervious section of most embankment dams, even those without exceptional differential settlement.*
>
> *In any event, as discussed earlier, the experience with dam behavior has shown practically no trouble with erosion of concentrated leaks through dam earth cores where there were reasonable downstream filters. As a result of this experience there has been less concern about differential settlement. The trend among many specialists, which I believe is justified . . . and in the correct direction toward both conservatism and economy, is to reverse the previous emphasis and consider that the downstream filter is the primary line of defense, and the other design measures to reduce the differential settlement are of secondary importance.*

Design of Small Dams, Third Edition, 1987 (Reclamation 1987a) also includes recommendations on chimney filter/drainage zones. This standard reference has been used extensively for guidance by embankment designers since it was published in 1960. The third edition (1987) is the source of the following citation. The citation emphasizes that a chimney filter is an important design element when embankment soils are erodible or contain potential flow paths.

> *Recently, to avoid construction defects such as loose lifts, poor bond between lifts, inadvertent pervious layers, desiccation, and dispersive soils, inclined filter drains in combination with a horizontal drainage blanket have become almost standard. Because drainage modifications to a homogeneous section provide a greatly improved design, the fully homogeneous section should seldom be used.*

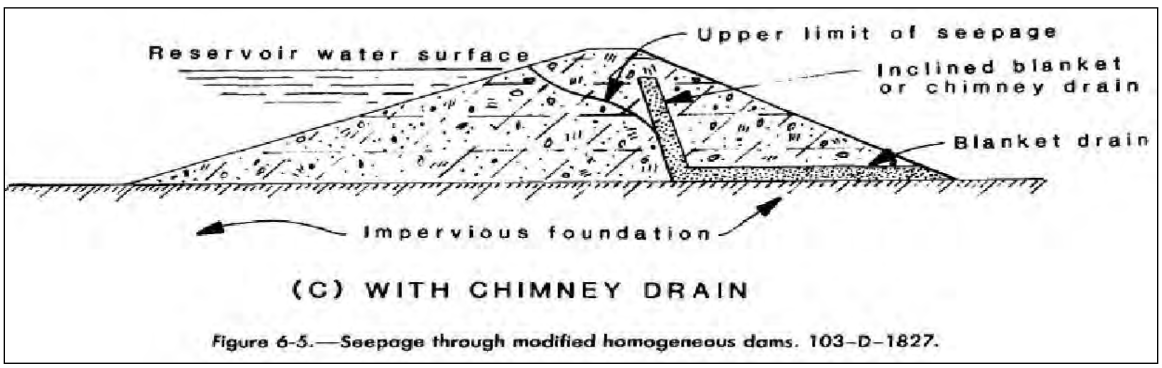

Figure L-29-1. Illustration from Design of Small Dams, Third Edition, 1987.

Another source for recommendations on chimney filters is the U.S. Army Corps of Engineers EM 1110-2-2300 (2004), General Design and Construction Considerations for Earth and Rock-Fill Dams. The following quotes from the manual emphasize the importance of including a chimney filter to protect against cracks that are likely to occur in a dam.

Chapter 7 on Embankment Design, Section 7-3, Cracking. p. 7-8.

> b. *Transverse cracking. Transverse cracking of the impervious core is of primary concern because it creates low paths for concentrated seepage through the embankment. Transverse cracking may be caused by tensile stresses related to differential embankment and/or foundation settlement. Differential*

settlement may occur at steep abutments, at the junction of a closure section, at adjoining structures where compaction is difficult, or over old stream channels or meanders filled with compressible soils.

e. *Defensive measures. The primary line of defense against a concentrated leak through the dam core is the downstream filter (filter design is covered in Appendix B). Since prevention of cracks cannot be ensured, an adequate downstream filter must be provided (Sherard 1984). Other design measures to reduce the susceptibility to cracking are of secondary importance.*

McCook (1997) discusses conditions for which chimney filters should be considered as required in an embankment design. Dr. Ronald Hirschfeld (1995) was a well-known expert in embankment design and provided the following advice on chimney filters:

Even the most careful design and construction cannot be relied upon to produce a dam in which there are no details that may lead to piping. Therefore, every embankment dam should have a chimney drain and a blanket drain beneath the downstream shell, as discussed in Section 3.0.

The most important conclusions about designing, constructing, and inspecting dams to minimize the risk of piping are:

Every dam should have a first line of defense against piping, in the form of a chimney drain and a blanket drain under the downstream shell, unless the designer can determine that there is no unacceptable risk in eliminating one or both. Details of design and construction that will reduce the risk of piping should be incorporated in the design of every dam, as described in Section 2.0.

Link_030_Use of Blankets in Embankments

While useful for filtering purposes, blanket drains may not be an effective method of controlling embankment seepage problems (drainage) for two reasons:

1. Embankments often have anisotropic permeability, at least the core zone in the embankment. Anisotropic permeability occurs when the horizontal permeability of the embankment section is much higher than the vertical permeability. This is often referred to as the Kh/Kv ratio. Many designers assume the ratio of horizontal to vertical permeability of embankment is between 9 and 25. The reason for this anisotropy in embankment permeability is that embankments are constructed as a series of horizontal layers (compacted lifts). This layering effect creates anisotropic permeability. This causes the phreatic line in the embankment to potentially override the blanket drain. If this occurs, the downstream slope of the dam can become saturated. Saturation of the downstream slope causes maintenance problems in that mowing equipment may become mired. Sloughing of saturated soils may also occur if the downstream slope is not flat enough.
2. Blanket drains are ineffective in intercepting potential cracks in the dam, particularly hydraulic fracture cracks or cracks caused by arching in the embankment. Chimney filters that extend vertically into the embankment are required to address this problem. A blanket drain may lead to a false sense of security if embankment cracking is ignored.

Cedergren (1973) illustrated with sketches of flow nets in his textbook how ineffective blanket drains may be in controlling the phreatic line if the embankment soils are anisotropic. Figure L-30-1 is a sketch that illustrates why a blanket drain may not be effective in controlling a phreatic line in a highly anisotropic embankment and how it is not effective in addressing hydraulic fracture cracks in a dam.

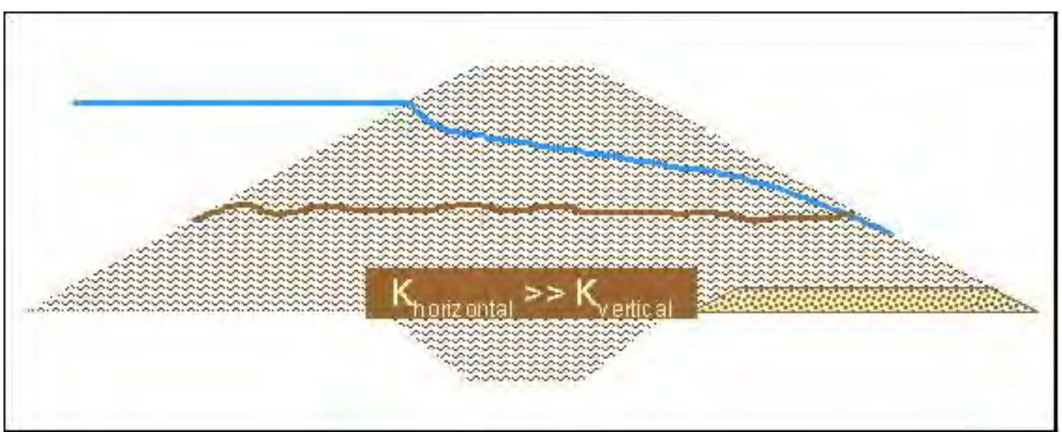

Figure L-30-1. Blanket drains are ineffective in controlling seepage in anisotropic embankments and do not address hydraulic fracture of the embankment.

Link_031_Foundation Trench Drain Inside Toe

Some historic designs have used a foundation trench drain placed well inside the toe of the embankment. Empirical guidelines were common at one time, governing placement of these drains inside the toe of the dam. The guidelines were based on an assumption that a foundation drain could be effective in controlling the phreatic line in the embankment, much like the argument made for a blanket drain affecting the location of the phreatic line.

Common guidelines were to place the foundation drain at a distance of from 60 to 80% between the downstream crest of the dam and the toe of the dam. Figure L-31-1 shows the placement of a toe drain in a location like this. For anisotropic embankments, and a normal pool level above midheight of the embankment, it is unlikely that a foundation drain alone (without an embankment chimney filter) will be effective in controlling the phreatic surface in the dam. It is likely that the downstream slope would become wet and difficult to maintain even if sloughing is not a problem.

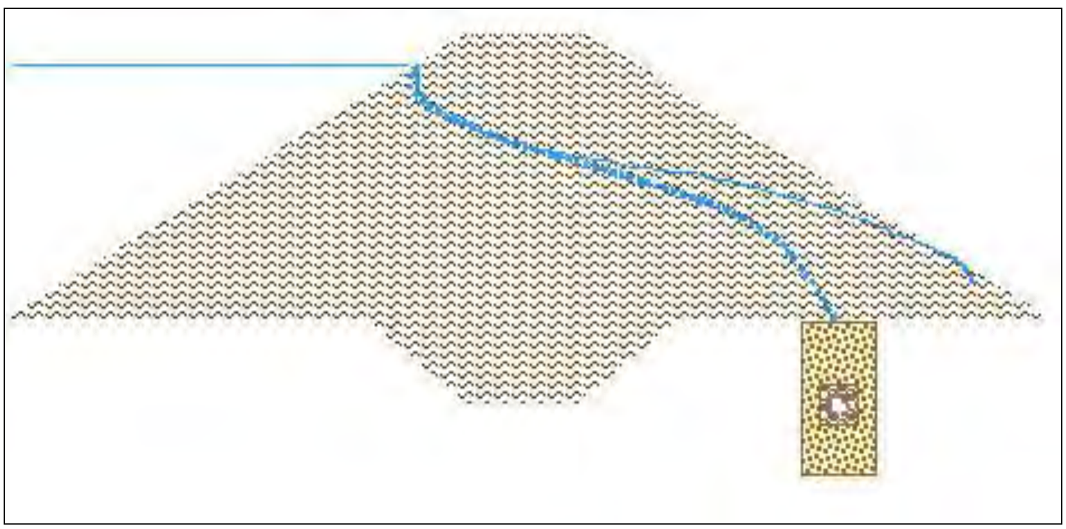

Figure L-31-1. Foundation trench drain placed inside the downstream embankment section in an attempt to both collect foundation seepage and lower the phreatic line. In anisotropic embankments, this dual function is not likely to be successful.

Another problem with placing drains well inside the footprint of an embankment is the difficulty in accessing the drain if it needs to be cleaned or replaced. This problem was dramatized with the Fern Ridge case history where the downstream slope of the dam had to be removed while the reservoir was maintained to replace a drain pipe and filter system. Figure L-31-2 shows the construction at Fern Ridge.

Figure L-31-2.—Foundation trench drain placed inside the downstream embankment section in an attempt to both collect foundation seepage and lower the phreatic line. At Fern Ridge, this dual function was not successful.

Link_032_Dewatering Considerations

Excavating foundation trench drains deep enough to intercept flow in anisotropic foundations may be difficult when a high water table occurs. A high groundwater table almost negates the use of rectangular configuration foundation trench drains because of the caving of the side slopes that occurs before drain material can be installed correctly. Trapezoidal trenches may be more stable, particularly if the side slopes are designed considering the effect of seepage forces on safety factors. Commonly, in sands with low clay content or silts, side slopes of at least 3.5H: 1V are required to excavate below the water table. An alternative is using dewatering wells or trench sump pumps to drawdown the water table and permit excavation of the trench. Figure L-32-1 shows a vacuum well point system that is very effective for drawing down the water table in sands and some silts.

Figure L-32-1.—Well points used to dewater excavation below the water table.

Link_033_Blanket Drain Overview

Some historic designs have used a blanket drain rather than a trench drain for control of foundation seepage. A blanket drain may be used based on an assumption that the blanket drain could be effective in controlling the phreatic line in the embankment as well as protecting the foundation from backward erosion piping or internal erosion. As previously noted, a blanket drain may not be effective in controlling the phreatic line in the embankment if the dam is anisotropic in permeability.

A foundation blanket drain is preferred over a trench drain when the foundation has preferential flow paths and a wide contact area is needed to intercept the maximum number of cracks. It is also a beneficial feature downstream of grouted bedrock to collect any potential crack flow missed by the grouting.

The best example of the effective use of a blanket drain is when it is placed on jointed bedrock. The drain needs to contact multiple joints to provide adequate pressure relief and capacity. A blanket drain placed over the downstream one-third of the dam, or under the shell of the dam in the case of a zoned fill, adds protection from any internal erosion that otherwise might occur from flow in minor ungrouted cracks in the bedrock. The advantage of a blanket drain is its larger contact surface area with the bedrock compared to a trench drain. Widths for blanket drains may be selected based on the average spacing of joints that a designer thinks are necessary to contact and gather seepage.