RECLAMATION
Managing Water in the West

Design Standards No. 13

Embankment Dams

Chapter 2: Embankment Design
Phase 4 (Final)

U.S. Department of the Interior
Bureau of Reclamation

December 2012

Mission Statements

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

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

Design Standards Signature Sheet

Design Standards No. 13

Embankment Dams

DS-13(2)-10: Phase 4 (Final)
December 2012

Chapter 2: Embankment Design

Chapter Signature Sheet
Bureau of Reclamation
Technical Service Center

Design Standards No. 13

Embankment Dams

Chapter 2: Embankment Design

DS-13(2)-10:[1] **Phase 4 (Final)**

Chapter 2 – Embankment Design is an existing chapter (last revised June 1992) within Design Standards No. 13 and was revised as follows:

- Minor revisions to reflect current terminology
- Minor revisions to reflect latest state-of-practice
- Added photographs and drawings

[1] DS-13(2)-10 refers to Design Standard No. 13, chapter 2, revision 10.

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Chapter 2

Embankment Design

2.1 Introduction

2.1.1 Purpose

The purpose of this chapter is to give basic guidance for the design of embankment dams within the Bureau of Reclamation (Reclamation).

2.1.2 Scope

Design procedures and concepts, with direction to appropriate chapters within Design Standard No. 13 - Embankment Dams for specific methods or analyses, are presented for both earth and rockfill dams. These guidelines are limited to design procedures for rolled-fill type construction. This type of placement is now used almost exclusively in the construction of embankment dams, to the exclusion of semi-hydraulic and hydraulic fills and dumped rockfills. The focus of this chapter is on the design of new embankment dams, although Section 2.4, "Evaluating and Modifying Existing Embankment Dams," discusses applicability to existing embankment dams.

2.1.3 Deviations from Standard

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

2.1.4 Revisions of Standard

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

2.1.5 Applicability

These standards apply to all embankment dams (earth or rockfill dams) designed by the Reclamation.

2.2 Earthfill Dams

2.2.1 Origin and Development

Embankments for the storage of water for irrigation, as attested to both by historians and archaeologists, have been used since the early days of civilization. Some of the structures built in antiquity were of considerable size. One earthfill dam that was 11 miles long, 70 feet high, and contained approximately 17 million cubic yards of embankment was completed in Ceylon (Sri Lanka) in the year 504 B.C. Today, as in the past, the earthfill dam continues to be the most common type of dam, principally because its construction involves using materials obtained from required excavation and locally available earth and rock materials obtained from borrow areas located near the damsite.

Until modern times, embankment dams were designed based on experience and precedence (empirical means). However, the engineering literature is replete with accounts of failures [1, 2, 3] of embankment dams. These failures produced the realization that totally empirical means must be replaced, or at least supplemented, by analytical engineering procedures in both design and construction. One of the first to suggest that the slopes for earthfill dams be selected based on analytical procedures was Bassell in 1907 [4]. However, little progress was made on the development of rational design procedures until the 1930s. The rapid advancement of the science of soil mechanics and geotechnical engineering since then has resulted in the development of greatly improved procedures for the design of embankment dams.

These procedures include:

- Thorough preconstruction investigations of foundation conditions and materials for construction
- Application of engineering analyses and experiences to design
- Carefully planned and controlled methods of construction
- Carefully planned and designed instrumentation and monitoring systems

Threaded throughout the Plan, Design, Construct, Operate, and Maintain process for an embankment dam is the philosophy that design is not complete until the dam is accomplishing its purpose and has proven itself safe through several cycles of operation.

As a result of analytical engineering procedures, a few embankment dams have been constructed to heights greater than 1,000 feet above their foundations, and hundreds of large rolled earthfill dams have been constructed in the past 50 years with a good success record.

2.2.2 General Comments on Earthfill Dams

2.2.2.1 Selection and Types of Earthfill Dams

The selection of type of dam is discussed in Design Standard No. 13, Chapter 1, "General Design Standards." When this procedure leads to the selection of an earthfill dam, a further decision must be made as to the type of earthfill dam: diaphragm, homogeneous, or zoned. An earthfill dam is designed considering the topographic and foundation conditions at the site and using available construction materials.

2.2.2.1.1 Diaphragm Embankments

In this type of section, the bulk of the embankment is constructed of pervious material (sand, gravel, or rock), and a thin diaphragm of impermeable material is provided to form the water barrier. The position of this impervious diaphragm may vary from a membrane placed on the upstream face to a centrally located diaphragm. The diaphragm or membrane may consist of asphaltic concrete, reinforced concrete, metal, compacted earthfill, or geomembrane. If compacted earthfill is used, the diaphragm is typically called a thin core. Internal diaphragms have the disadvantage of not being readily available for inspection or emergency repair if they are ruptured due to a material flaw or settlement of the dam or its foundation.

Examples of various materials that have been used to provide a diaphragm water barrier include:

1. Geomembrane on upstream face - Warren H. Brock Reservoir, California
2. Internal geomembrane – Reach 11 Dikes, Arizona
3. Concrete cutoff wall - Tieton Dam, Washington
4. Other cutoff walls such as secant pile (Lake Tahoe Dam) and sheet pile (Fourth Creek Dam, U.S. Bureau of Indian Affairs [BIA])
5. Plastic concrete cutoff wall – Meeks Cabin Dam, Wyoming
6. Fiber reinforced polymer – Coquille Dams (BIA), Oregon
7. Cement bentonite cutoff wall – Diamond Creek Dike, Wyoming
8. Steel sheetpile cutoff wall – Lake Wolford Dam, California

9. Steel plates on upstream face – El Vado Dam, New Mexico

10. Concrete-faced rockfill – Exchequer Dam, California

11. Asphaltic concrete-faced embankment – Montgomery Dam, Colorado

If the bulk of material comprising the diaphragm-type dam is rock, the dam is classified as a rockfill dam. The design of rockfill dams is discussed in Section 2.3, "Rockfill Dams."

2.2.2.1.2 Homogeneous Embankments

A significant number of embankment dams have been designed and constructed, and the majority of these dams may well be homogenous embankments. Similarly, many of Reclamation's dams are homogeneous. In general, embankment dam design philosophy has changed from minimizing seepage (with a wide homogeneous cross section) to controlling seepage by incorporating filter and drainage elements. As such, purely homogeneous embankments are not recommended when designing/constructing new dams, especially high hazard dams.

A purely homogeneous type of dam is composed of a single kind of material (except for the slope protection). The material comprising the dam must be sufficiently impervious to provide an adequate water barrier. Soils meeting this requirement generally have shear strengths such that the slopes of the dam must be relatively flat for stability. To avoid sloughing, the upstream slope must be flat enough to maintain stability if rapid drawdown of the reservoir after long-term storage is anticipated. The downstream slope must be flat enough to provide embankment stability when the reservoir is filled and the bulk of the dam becomes saturated. For a completely homogeneous section on an impervious foundation, seepage will emerge on the downstream slope regardless of the embankment slope and the permeability of the embankment material if the reservoir level is maintained for a long period of time. Under such conditions, the downstream slope will eventually be affected by seepage to a theoretical height of roughly one-third the depth of the reservoir pool, as shown on figure 2.2.2.1.2-1(A). However, in practice, whether or not seepage exits on the downstream face depends on the permeability of the foundation and embankment materials, as well as reservoir operations.

Although formerly very common, the purely homogeneous section-type dam has been replaced by a modified homogeneous section in which internally placed pervious materials control seepage and the saturation (phreatic surface) within the dam, thus permitting steeper slopes. The modification of the homogeneous type of section to include drainage features provides a greatly improved design. The modified homogeneous type of dam is applicable in localities where readily available soils show little variation in permeability, and soils of contrasting permeabilities are available only in minor amounts or at considerably greater cost.

Figure 2.2.2.1.2-1 shows the effect of providing drainage at the downstream toe in a modified homogeneous dam.

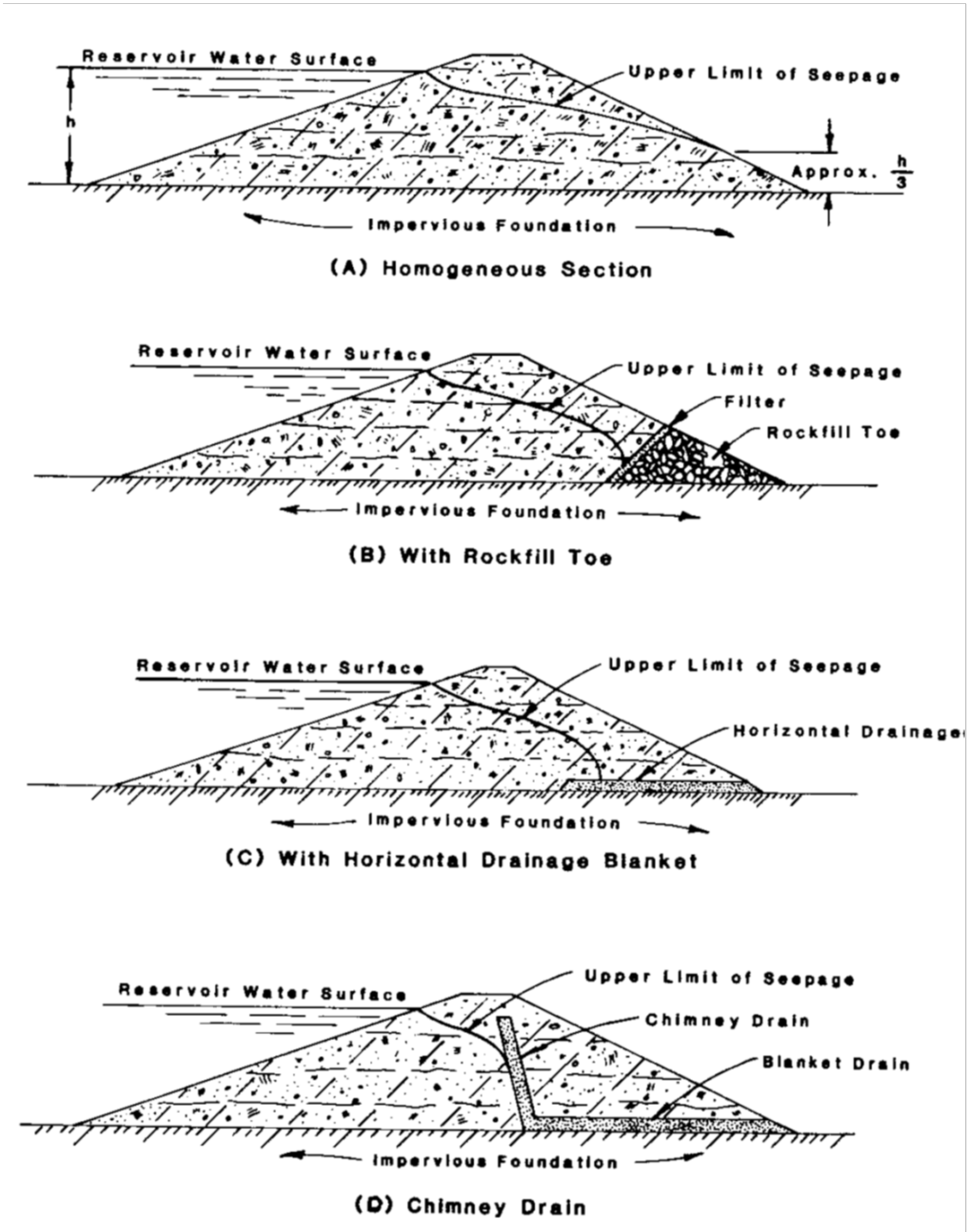


Figure 2.2.2.1.2-1. Effects of drainage.

Theoretically, rock toes or drainage blankets, as shown on figures 2.2.2.1.2-1(B) and 2.2.2.1.2-1(C) will lower the phreatic surface. However, these features are not designed to intercept seepage or leakage (and possible internal erosion) that can progress along cracks that may form after fill placement or to intercept more pervious, or loosely compacted, or poorly bonded lifts that might inadvertently occur during construction. Fundamentally, this lack of an internal filter/drainage feature to prevent internal erosion is probably the most significant deficiency with this type of dam. Another consideration is the fact that embankments with soils placed in layers are inherently much more pervious horizontally than vertically, causing a tendency for seepage to advance more rapidly and further horizontally. That ratio is difficult to predict, which makes it difficult to predict where seepage will exit on the slope. Figure 2.2.2.1.2-1(D) shows a chimney filter/drain that is designed to intercept these flows and mitigate potential deficiencies. In recent years, use of the chimney filter/drain has become standard practice in all homogeneous type dams and should be included in all Reclamation dams unless very specific circumstances preclude the need. The drainage and filter layers must be designed to meet filter requirements with surrounding fill or foundation materials.

Another method of improving and collecting drainage is the installation of pipe drains. These are normally used in conjunction with the horizontal drainage blanket or toe drain. Pipe drains should only be located in areas where they can be inspected, maintained, and accessed for repair without affecting embankment slope stability.

A homogeneous section should never be used if the available materials are dispersive and erodible, such as silts and fine sands, or if they are subject to moderate to severe cracking because of desiccation or high seismicity. Soils should always be tested for these characteristics.

2.2.2.1.3 Zoned Embankments

Compared to a modified homogeneous dam, zoned dams (such as Ridgway, McPhee, Jordanelle, and New Waddell Dams) are usually constructed in areas where several material types are available, such as clays, silts, sands, gravels, and rock. Zoned embankments take advantage of the availability of various materials by placing different materials in various zones so that their best properties are used most beneficially, and their poor properties are mitigated. A zoned earthfill dam typically has a central impervious core flanked by upstream transition zones, downstream filters and drains, and then outer zones or shells composed of gravelfill, rockfill, or random fill, which are considerably stronger than the core. Depending on the gradation of available materials, transition zones may not be necessary. The shells support and protect the impervious core, transition zones, filters, and drains; the upstream pervious zone provides strength for stability against rapid drawdown; and the downstream zone provides strength to buttress the core and filters so that steeper (more economical) slopes can be used. The upstream transition zone, if necessary because of a very pervious shell, provides protection against internal erosion or washout of the core during rapid drawdown, and protection against cracking of the core. The downstream filters and drains control seepage and leakage and prevent sediment transport through any cracks in the central impervious core. The dam is considered to be a thin core dam if the impervious zone has a horizontal width less than 10 feet at any elevation below normal reservoir level, or if it has a ratio of hydraulic head to horizontal width of 2.0 or greater. This ratio should not be greater than 4 without special analyses and provisions to control seepage and high seepage exit gradients through the impervious core and its foundation.

The shells and transition zones preferably consist of sand, gravel, cobbles, or rock, or mixtures of these materials. The impervious core is constructed from more impervious fine-grained soils such as silts, clays, sandy silts, sandy clays, and gravelly clays, or mixtures thereof. Although not as desirable, fat clays and gravelly silts have also been used for the impervious zone. Gravelly clays, sandy clays, and lean clays are the most desirable impervious materials. The filters and drains are generally processed from available sands and gravels and must meet filter criteria with surrounding materials (see Design Standard No. 13, Chapter 5, “Protective Filters”). Chapters 4, 5, 7, 8, and 9 of Design Standard No. 13 discuss the design of the various zones.

A dam with an impervious core of moderate width composed of strong material and with pervious outer shells may have relatively steep outer slopes, limited only by the strength of the foundation material, the stability of the embankment itself, and maintenance considerations. Conditions that tend to increase stability may control the choice of a design cross section even if a longer haul is necessary to obtain such embankment materials.

If a variety of soils are readily available, a zoned embankment will usually be chosen because it is generally superior for both stability and seepage performance. Zoned dams (as discussed later) also afford better ability to use material in the embankment section from required excavation. Materials that are closest to the dam and require the least processing should be used for the best economy.

2.2.2.2 Design Data

The data required from investigation of foundations and sources of construction materials for the design of an embankment dam are discussed in Design Standard No. 13, Chapter 12, “Foundation and Earth Materials Investigation”; *the Earth Manual* [5 and 6], and the *Engineering Geology Field Manual* [7 and 8]. The extent of required data and methods of obtaining the data will be governed by the nature of the project and the purpose of the design (i.e., whether the design is intended as a basis for a cost estimate to determine project feasibility, for construction, or some other purpose). The extent of investigations of foundations and sources of construction material will also be governed by the complexity of the site conditions.

2.2.2.3 Criteria for Design

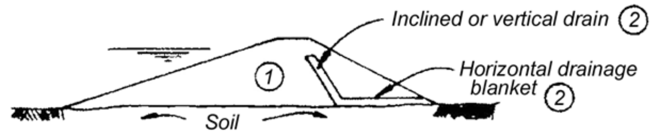
The basic objective of design is to produce a satisfactory functional structure at a minimum total cost. Consideration must be given to maintenance requirements so that any cost savings achieved by the selection or elimination of certain design features will not result in excessive maintenance costs. Maintenance costs vary with the provisions of upstream and downstream slope protection, availability of materials for future maintenance and repair, drainage features, and the type of appurtenant structures and mechanical equipment.

To achieve minimum cost, the dam should be designed for maximum use of the most economical materials available, including materials which must be excavated for the dam's foundation and for appurtenant structures. Figure 2.2.2.3-1 illustrates the types of earthfill dams.

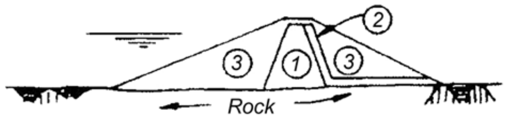
An earthfill dam must be safe and stable during all phases of construction and operation of the reservoir. This requires defensive design measures and usually some redundancy because of the potential hazard to the public from most dams. For example, control of seepage and leakage requires the use of an impervious zone of some kind within the embankment and within cutoff trenches excavated through pervious zones within the foundation. Additionally, filters and drainage features are required to control seepage or leakage that may find its way through impervious zones, and to protect against internal erosion. In earthquake regions, the filters and drains are designed to increased widths to provide for the potential occurrence of cracking or displacement of the embankment during an earthquake. Toe drains are added to provide seepage control and, in pervious foundations, relief wells are sometimes used to control seepage or reduce pore pressures deeper in the foundation.

An earthfill dam designed to meet the requirements listed in Design Standard No. 13, Chapter 1, "General Design Standards," will safely fulfill project objectives provided that proper construction methods and control are achieved. The design procedures and guidelines necessary to meet the requirements of these criteria are provided in various other chapters of *Design Standard No. 13. Embankment Dams*.

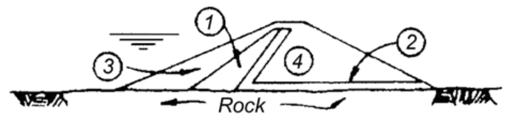
EARTHFILL DAM SECTIONS



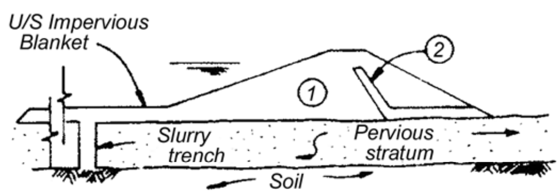
a. Homogenous dam with internal drain on impervious foundation



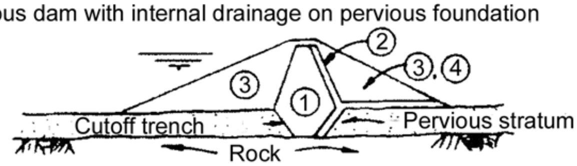
b. Central core, zoned dam on impervious foundation



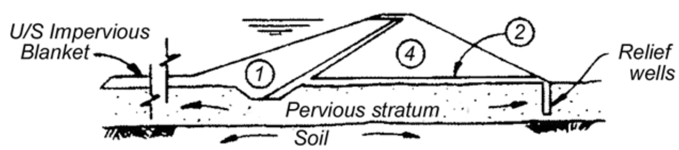
c. Inclined core, zoned dam on impervious foundation



d. Homogenous dam with internal drainage on pervious foundation



e. Central core, zoned dam on pervious foundation



f. Zoned dam with upstream impervious zone on pervious foundation

Note: Sections are modified from those presented in Fig. 1-1, EM 1110-2-2300, Dept. of the Army, Corps of Engineers, May 10, 1982

LEGEND:

1 - Zone 1, impervious soil

2 - Zone 2, filter drain material (may require a two-stage system - usually processed sands and gravels)

3 - Zone 3, pervious soil (sands and gravels)

4 - Zone 4, random soil (requires adequate engineering properties, but plasticity and gradation are less critical considerations)

Figure 2.2.2.3-1. Types of earthfill dams.

2.2.2.4 General Construction Methods

The designer assumes that certain construction methods are used, such as placement in lifts, and that equipment such as rollers are used for compaction of the embankment. The success of the design depends on implementation of these assumptions; therefore, it is necessary to monitor construction to ensure that appropriate equipment and construction methods are used. These considerations are discussed in detail in Design Standard No. 13, Chapter 10, “Embankment Construction.”

2.2.3 Foundation Design for Earthfill Dams

2.2.3.1 General

The term “foundation,” as used herein, includes both the valley floor and the abutments upon which the embankment will be built. A foundation for an earthfill dam has two essential requirements: (1) it must provide stable support for the embankment under all conditions of saturation and loading, and (2) it must provide sufficient resistance to seepage to prevent internal erosion or excessive loss of water.

Although the foundation is not actually designed, certain provisions for treatment of the foundation are provided in designs to ensure that the essential requirements will be met. Such measures may include excavation of unsatisfactory materials, foundation grouting, material densification, use of filters, and surface treatment measures such as shaping, slush grouting, and dental concrete. Each foundation presents its own separate and distinct problems that require corresponding special treatment and preparation. Various methods for stabilizing weak foundations, reducing seepage in permeable foundations, shaping to reduce differential settlement to acceptable levels, and types and locations of devices for intercepting underseepage must be adapted to local conditions.

Surveys and compiled statistics vary [1, 2, 9, 10], but it appears that between 10 and 20 percent of embankment dam failures, and close to 50 percent of incidents at embankment dams, can be attributed to the foundation. These statistics indicate the importance of understanding the foundation. The foundation must be adequately explored to characterize its properties. The data from the exploration program is interpreted by engineering geologists and must reveal subsurface conditions to permit safe and economical design of foundation treatment. The exploration program should be a continuing process (see *Final Design Process* [11]) that begins with inception of the project and continues through construction. The program should build on data from previous investigations as the design progresses. It is guided and adjusted by geologic interpretation of the data. The accuracy of the geologic picture should be continuously evaluated as additional data become available during all phases of design and construction.

Theoretical solutions based on principles of soil and rock mechanics can be obtained for problems involving seepage, settlement, and stability of foundations. However, it is difficult to model embankments and foundations precisely because it is difficult to determine strengths and permeabilities, and their variability, accurately. Therefore, sound engineering judgment plays an extremely important role in applying theory to practice, as does the incorporation of redundant design features (multiple lines of defense).

Because certain types of treatment are appropriate for particular foundations, they are grouped into three main classes according to their predominant characteristics:

1. Foundations consisting of rock
2. Foundations consisting of coarse-grained material (sand and gravel)
3. Foundations consisting of fine-grained material (silt and clay)

However, many foundations are comprised of materials which originate from various sources such as river alluvium, glacial outwash, talus, and other processes of erosion, disintegration, and deposition. They are not characterized by a single material, but, rather, by a complex combination of structural arrangement and physical characteristics of their constituent materials. Foundation deposits may be roughly stratified, containing layers of clay, silt, sand, and gravel; or they may consist of lenticular masses, pockets, and channels of the various materials without any regularity of occurrence and of varying extent and thickness. In spite of this, the character of a foundation can be revealed adequately by geologic exploration. Once the geology is properly understood, design and construction techniques can usually be employed to achieve an adequate and safe embankment foundation.

Analyses and construction techniques required for the different types of foundations are discussed in specific chapters of *Design Standard No. 13, Embankment Dams*.

The foundation of a dam will usually consist of a combination of the three main types of foundations listed previously. For example, the stream portion of the foundation is often a sand-gravel foundation, while the abutments are rock which is exposed on steep slopes and may be mantled by deposits of clay or silt on the gentler slopes. Therefore, the design of any one dam may involve a variety of foundation design considerations.

2.2.3.2 Rock Foundations

2.2.3.2.1 General

Foundations consisting of rock are generally considered more competent than soil foundations. Even foundations of weaker rock are generally preferred over soil foundations. The preference for a rock foundation is undoubtedly justified where the rock mass is generally homogeneous and competent; however, because rock foundations can contain numerous joints, faults, and other discontinuities, they need to be carefully investigated to ensure that they are competent.

Rock foundations containing claystones, siltstones, and shales are sometimes particularly treacherous from a stability standpoint. Weak zones, seams, or layers must be located so that foundation stability under the applied load of embankment and reservoir can be adequately analyzed. For example, many sedimentary foundations contain weak layers of clay that may be only millimeters thick. If these layers are not carefully searched for during exploration work, and located and accounted for during design and construction, they can lead to stability failures.

If clayey portions of rock foundations are erodible or dispersive, there is also a threat of internal erosion. In addition, rock foundations containing faults, fractures, or soluble zones (such as gypsum) can cause serious seepage or leakage problems. Potential paths of excessive seepage or leakage must be located and adequately treated to control seepage that could lead to internal erosion at the interface between the foundation/embankment contact, to prevent uneconomical loss of water, and to ensure that provisions are made to adequately control hydraulic pressures in the foundation. An untreated solution channel, fault zone, volcanic dike or sill, or fracture zone can transmit essentially full reservoir head to the downstream area of a dam where drainage features may be overloaded by unanticipated pressure and flow. In such cases, instability may result from excessive internal erosion or uplift pressure.

2.2.3.2.2 Underseepage

Permeability of rock foundations needs to be determined to estimate seepage losses and gradients. Secondary permeability (not primary) is much more important in rock for these computations. Field testing is considered the best way to obtain values of secondary permeability. Such testing must be performed at various locations representing different geology and site conditions. Design Standard No. 13, Chapter 8, "Seepage," contains additional discussion.

2.2.3.2.3 Foundation Surface Treatment

Foundation rock surfaces, against which fill will be placed, must be properly treated to ensure that fractures, fault zones, steep faces, rough areas, weathered zones, etc., do not lead to seepage and internal erosion at the foundation/ embankment contact. Treatment of deficient foundation zones is especially critical for impervious core foundations, and often critical in the filter and drainage zones immediately downstream of the impervious zone.

All loose and overhanging rock must be removed from the abutments to create surfaces that are suitable for embankment placement and compaction; rock slopes should not be steeper than 0.5:1 H:V (horizontal:vertical) and preferably flatter. Where flattening of the rock slopes or overhangs is not practicable, the slopes may be shaped by the use of dental concrete.

If the bedrock is a shale that slakes in air, it may be necessary to excavate several feet into bedrock to remove the surface disintegration just before placement of the embankment; in more durable rock types, little excavation into the bedrock is usually necessary. Fractured rock should be treated by slush grouting.

Foundation surface treatment requirements are discussed in Design Standard No. 13, Chapter 3, “Foundation Surface Treatment.”

2.2.3.2.4 Grouting

Preliminary designs and estimates for a storage dam should include foundation grouting, which includes curtain grouting and blanket grouting. Foundation grouting is a process of injecting cementitious slurries under pressure into the underlying formations through specially drilled holes for the purpose of filling joints, fractures, fissures, bedding planes, cavities, or other openings. Grouting is generally used to reduce erosive leakage, excessive uplift pressure, and high water losses through the foundation rock. This use generally applies to the design of new dams, but grouting can also be used as a remedial measure to help control seepage at existing dams.

Foundation grouting is an engineering process that must be designed and planned for in the office by an experienced designer, grouting specialist, and geologist and then executed by competent field personnel who have the required experience. The foundation grouting design should be based on the site geology, exploration, seepage analyses, long-term maintenance, and the value of any water loss within the foundation. The design and specifications must be flexible to adjust to the conditions observed in the field during grouting. The engineering process for foundation grouting is not complete until the last hole is grouted.

Curtain grouting is probably the most common method of foundation seepage reduction used beneath dams. This method consists of drilling holes into the foundation bedrock at some regular spacing along a line or lines parallel to the dam axis and normal to the seepage flow direction. Cement grout is then pressure injected into the drilled hole to fill joints, fractures, fissures, bedding planes, cavities, or other openings within the bedrock. Unless special geologic conditions dictate otherwise, general practice for Reclamation grout curtains is to grout the foundation to a depth below the surface of the rock equal to 0.5 to 1.0 times the reservoir head which lies above the surface of the rock. For zoned embankment dams with a central impervious core, the grout curtain is typically located slightly upstream of the midpoint of the base of the core. If the grout curtain is placed closer to the upstream toe of the core, then high gradients may exist from the embankment into the foundation. If the grout curtain is placed closer to the downstream toe of the core, then high gradients may exist from the foundation into the core.

The grout mixes used for Reclamation grout curtains rely on the "optimum mix" theory. Reclamation foundation grouting practice uses thinner starting mixes than used around the world, typically starting at 5:1 (water to cement ratio, by volume) with super-plasticizer. The grout mix is gradually thickened until the optimum mix is found for each grouting stage. Using the optimum mix theory that allows for relatively larger grout travel distances upstream and downstream of the grout curtain, curtain grouting at Reclamation dams typically use a single-line grout curtain within an 80-foot closure pattern consisting of primary, secondary, tertiary, and quaternary holes spaced on 10-foot intervals. Additional split spacing (quinary and senary holes) can be used if closure is not obtained. In some cases where poor foundation conditions are encountered, such as highly fractured rock or soluble rock, a multiple line grout curtain may be necessary.

A grout curtain should never be relied on as the singular provision to reduce seepage and related uplift pressure to the extent that downstream drainage or pressure relief features are reduced or eliminated.

In cases where large zones of fractured rock lie at the foundation contact, blanket grouting is often used to reduce leakage into the fractured zone and to provide a firm foundation for the dam. Blanket grouting is normally used only beneath the impervious zone of the embankment. This type of grouting is very valuable in preventing erosive seepage or flow through rock fractures near the impervious zone/foundation contact within a rock foundation, and it should almost always be included in the design. Blanket grouting is generally used in combination with curtain grouting and is typically performed prior to curtain grouting. Blanket grouting holes are generally 20 to 30 feet deep and are arranged on a grid pattern with primary holes spaced at 20 feet. Spacing of additional grout holes between the primary holes is reduced as necessary by a split spacing method to achieve closure between the primary holes.

Grouting methodology is discussed in Design Standard No. 13, Chapter 15, "Foundation Grouting," and in several other publications [12, 13, and 14].

2.2.3.2.5 Cutoffs

In some very pervious rock foundations such as porous sandstone or those containing soluble layers such as limestone or gypsum, it may be appropriate to provide cutoffs through pervious strata to control seepage and reduce solutioning. Cutoffs are sometimes advisable through upper layers of weathered or broken foundation rock. Shallow cutoffs are usually accomplished by cutoff trenches excavated with sloping sides and backfilled with compacted earthfill with proper filters as necessary. To provide a sufficient thickness (width) of impermeable material and an adequate contact with the rock or other impervious foundation stratum, the bottom width of the cutoff trench should increase with an increase in reservoir head. An adequate width for the cutoff trench at a small dam may be determined by the formula:

$$w = h - d$$

where: w = bottom width of cutoff trench
h = reservoir head above ground surface
d = depth of cutoff trench excavation below ground surface

In any case, a minimum bottom width of 20 feet should be provided so that excavating and compacting equipment can operate efficiently in trenches, as well as allowing space for any dewatering or unwatering facilities.

For larger embankment dams, and particularly for those where erodible soils are present or high seepage is expected, seepage analyses and/or consideration of expected gradients should be considered to determine the appropriate bottom width of cutoff trenches.

Where deep cutoffs are required, thin foundation cutoffs, such as a concrete diaphragm wall, may be more economical. Design Standard No. 13, Chapter 16, “Cutoff Walls,” discusses the design of thin foundation cutoffs.

2.2.3.2.6 Filters and Drains

Filters and drains are the primary features for collecting and controlling seepage that passes through and under dams on rock foundations. Even though a rock foundation may be grouted and cutoffs provided, appropriate filters and drainage are necessary to collect seepage and reduce uplift and seepage pressures in the areas downstream of the impervious zone and beyond the downstream toe of the dam. This is a necessary design feature that provides defense against unforeseen events such as unknown foundation discontinuities, foundation fracturing caused by earthquakes, or construction deficiencies that may occur in grout curtains and cutoffs. Drainage blankets, toe drains, toe trenches, and relief wells (not very effective in most rock formations) should be used individually, or in combination as necessary, to ensure control of seepage. Design Standard No. 13, Chapter 5, “Protective Filters,” and Chapter 8, “Seepage,” discuss the design of these features.

2.2.3.3 Sand and Gravel Foundations

2.2.3.3.1 General

Often, the foundations for dams consist of alluvial deposits comprised of relatively pervious sands and gravels overlying more impervious geological formations. The pervious materials may range from fine sand to openwork gravels, but more often they consist of stratified heterogeneous mixtures. Generally, sand and gravel foundations have sufficient strength to adequately support loads induced by the embankment and reservoir; however, the dam's stability must be verified by adequate exploration, testing, and analyses. Knowledge of the geologic deposition process can aid in determining the potential for the presence of low strength zones.

Three basic problems are generally found in pervious foundations: (1) the amount of underseepage, (2) the pressures exerted by the seepage, and (3) the potential for internal erosion. The type and extent of treatment justified to decrease the amount of seepage will be determined by the purpose of the dam, the tolerable seepage losses, and the necessity for making constant reservoir releases to serve senior water rights or to maintain a live stream for fish, etc. Loss of water through underseepage may be of economic concern for a storage dam but of little consequence for a detention or flood control dam. Economic studies of the value of the water and the cost of limiting the amount of underseepage are required, in some instances, to determine the extent of foundation treatment. However, adequate measures must be taken to ensure the safety of the dam against failure due to internal erosion, or instability caused by seepage and uplift pressures, regardless of the economic value of the seepage.

An additional problem may exist in foundations consisting of low-density sands and gravels. The loose structure of saturated sands and gravels is subject to collapse (liquefaction) under the action of earthquake loading. Although loose sand may support sizable static loads due to point-to-point contact of the sand grains, a vibration or shock may cause readjustment of the grains into a more dense structure. Because drainage cannot take place instantaneously, part of the static load formerly carried by the sand grains is transferred temporarily to the water, and the effective strength of the foundation may be greatly reduced, leading to failure. Foundations consisting of cohesionless sand and gravel of low density are suspect, and special investigations and analyses should be made to determine required remedial treatment. See Design Standard No. 13, Chapter 13, “Seismic Design and Analysis” for more information.

Some very loose sand foundations may also be collapsible under static loading. They sustain the load from construction of the embankment and then, during wetting or saturation during reservoir filling, they consolidate rapidly or “collapse.” These types of foundation soils must be identified and accounted for in the design.

Design methods for controlling settlement range from preconsolidation by wetting and preloading of the soil to densification procedures such as compaction piles, dynamic compaction, or removal of the soil. For major dam structures, removal is usually the preferred solution; however, the feasibility and cost of removal should be evaluated.

Design Standard No. 13, Chapter 9, “Static Deformation Analysis,” describes testing procedures and analytical methods for identifying and predicting consolidation in collapsible soils.

2.2.3.3.2 Underseepage

To estimate the volume of underseepage that may be expected, it is necessary to determine the coefficient of permeability of the pervious foundation. This coefficient is a function of the size and gradation of the coarse particles, the amount of fines, and the density of the mixture. In general, field tests are preferred over laboratory tests for estimating permeability. Three field test methods are typically used in determining the coefficient of permeability of foundations: (1) pump-out tests in which water is pumped from a well at a constant rate, and the drawdown of the water table is measured in wells placed on radial lines at various distances from the pumped well; (2) tests conducted by observation of the velocity of flow as measured by the rate of travel of a dye or electrolyte from the point of injection to a seepage discharge point; and (3) pumping-in tests in which water is pumped into a drill hole or test pit, and the flow rate is measured for a given head. There are also various laboratory test methods that are used to determine the coefficient of permeability, such as permeability and settlement tests, one-dimensional consolidation test, and falling head and constant head permeability tests. Most of these test methods are discussed in the *Earth Manual* [5 and 6]. Seepage analyses and control are discussed in Design Standard No. 13, Chapter 5, “Protective Filters,” and Chapter 8, “Seepage.”

2.2.3.3.3 Seepage Reduction and Control

Various methods of seepage reduction and/or control can be used, depending on the requirements for preventing uneconomical loss of water and the likelihood that the foundation will transmit water forces and pressures related to seepage, which can contribute to static instability and cause internal erosion, heave, or blowout. Cutoff trenches backfilled with compacted soil, cement-bentonite cutoff walls, soil-bentonite cutoff walls, soil-cement cutoff walls, concrete cutoff walls, upstream impervious blankets, or combinations of these methods have been used to reduce seepage flow and to help control related water pressures. Downstream drainage blankets, toe drains, drainage trenches, relief wells, or combinations thereof are used to collect seepage, thereby reducing related water pressures so that static instability, heave, blowout, and internal erosion are adequately controlled in the downstream zones of the foundation.

2.2.3.3.4 Filters and Drains

Typical drainage features include drainage blankets, toe drains, and drainage trenches. Horizontal drainage blankets may be incorporated into the downstream section of a dam or used to blanket the area immediately downstream from the toe of the dam to intercept and control water seeping and flowing from the foundation that may be under excess hydrostatic head. The purpose of these blankets is to permit free flow and dissipation of pressure without disruption of the foundation structure and loss of fine soil particles (internal erosion) that can lead to failure. Toe drains are commonly installed along the downstream toes of dams in conjunction with horizontal drainage blankets. The purpose of these drains is to collect the seepage discharging from the embankment and foundation and convey it to an outfall pipe, which usually discharges into either the spillway, outlet works stilling basin, or the river channel downstream of the dam. Drainage ditches containing perforated pipes surrounded with filter and/or drain material rather than French drains should be used to ensure adequate capacity to carry seepage flows. Drainage trenches backfilled with properly designed filter and drainage material may be used to intercept and control seepage in the shallower zones of the foundation to prevent internal erosion and blowout at the critical area in the vicinity of the downstream toe. Also, perforated pipes are usually used with toe trenches to collect and convey water from the toe trenches to discharge points. Design of these features and example drawings are discussed in Design Standard No. 13, Chapter 5, “Protective Filters,” and Chapter 8, “Seepage.” In conjunction with the installation of toe drains, seepage measurement instrumentation is usually provided to evaluate the performance of the dam. Design of instrumentation is discussed in Design Standard No. 13, Chapter 11, “Instrumentation.”

2.2.3.3.5 Relief Wells

Pressure relief wells are devices used to relieve water pressures deeper in foundations by intercepting seepage at a level below the ground surface that cannot economically be cut off or reached by drainage trenches. Relief wells are generally used to prevent uplift or blowout of impervious zones or layers that overlie much more permeable zones, but relief wells can also be used on close spacing to relieve pressures or intercept and safely control seepage in erodible materials such as fine sands. They are also useful for remedial treatment at existing dams to relieve high-pressure zones. Relief wells were used at Jamestown Dam, an existing Reclamation dam in North Dakota, to control seepage and high hydrostatic pressures in the foundation. Similar applications include Sanford Dam and Twin Buttes Dam in Texas, Foss Dam in Oklahoma, and Virginia Smith Dam in Nebraska. Design Standard No. 13, Chapter 8, “Seepage,” discusses this type of design.

2.2.3.3.6 Cutoffs

When used primarily for seepage reduction, the cutoff trench is preferably located beneath or upstream from the centerline of the crest of the dam. However, it should not be so far upstream that the cover of impervious embankment above the trench will fail to provide resistance to seepage at least equal to that offered by the trench itself. In addition, it should not be so far upstream that any future required drilling into the foundation through the cutoff from the crest for exploration, installation of instrumentation, grouting, etc., becomes difficult. The centerline of the cutoff trench should be kept parallel to the centerline of the dam across the canyon bottom or valley floor, but it should converge toward the centerline of the dam as it is carried up the abutments in order to maintain the required impervious embankment cover. Cutoff trenches can be partial or fully penetrating, depending on the properties of the foundation materials and the necessity to conserve water. However, a partially penetrating trench does not appreciably reduce the quantity of seepage unless it penetrates the most pervious strata, leaving only semi-pervious to impervious material below it, or unless it penetrates almost completely to an impervious zone in the foundation. Partially penetrating cutoff trenches are sometimes excavated for inspection of the upper part of the foundation during construction and to penetrate upper looser zones of the foundation.

Cutoff trenches may be classified into two general types: (1) compacted earthfill trenches (with sloping sides), and (2) flowable fill trenches (with vertical sides). Traditional compacted earthfill cutoff trenches are usually preferred. Sloping side cutoff trenches are excavated using loaders, backhoes, shovels, draglines, or scrapers and are backfilled with impervious materials which are compacted in the same manner as the impervious zone of the embankment. In addition, the downstream face of the excavated trench is usually protected by a filter. Thin flowable fill cutoffs are usually used where a deep cutoff is required and open cut excavation is not economical. These are generally constructed in slurry-stabilized trenches as earth-slurry backfill walls, concrete diaphragms, cement-bentonite walls, etc. A cement-bentonite wall was constructed at Reclamation’s existing A.V. Watkins Dam in Utah. Sections and pictures are presented in Design Standard No. 13, Chapter 1, “General Design Standards.” Design of thin foundation cutoffs is discussed in Design Standard No. 13, Chapter 8, “Seepage,” and Chapter 16, “Cutoff Walls.”

When embankment stability is an issue (for example, when liquefaction of foundation materials may be a concern), the cutoff trench serves not only to reduce seepage, but also to improve the foundation. Removal and replacement of low strength or potentially liquefiable foundation soils may require more than one “foundation trench” or an excavation that extends from nearly the upstream to downstream toe. It has been found that a cutoff trench upstream of the dam centerline is of benefit for seismic or rapid drawdown stability of the upstream slope. Jordanelle Dam in Utah features a near-complete removal of coarse-grained foundation soils to ensure that foundation liquefaction under large earthquakes on the Wasatch Fault does not compromise the seismic embankment stability.

2.2.3.4 Silt and Clay Foundations

2.2.3.4.1 General

Fine-grained soils are generally sufficiently impermeable to preclude the necessity of providing cutoff design features for underseepage and internal erosion. The main problem with these foundations is instability. In addition to the obvious danger of instability of foundations of saturated silts and clays, the designs must take into account the effect of reservoir-induced postconstruction saturation of the foundations on the dam and appurtenant works. Methods of foundation treatment are based on the soil type, the location of the water table, and the density of the soil.

Although all soils can be susceptible to erosional and internal erosion failure, certain clay soils are particularly vulnerable. These clays are called “dispersive soils” due to their tendency to disperse or deflocculate in water. The tendency for dispersion depends on variables such as clay mineralogy and chemistry of both the clay and dissolved water in the soil pores. Laboratory tests, such as the crumb, double hydrometer, pinhole, and soil chemistry tests, can be used to identify dispersive soils. Dispersive soil can be used in the impervious zone of the embankment provided that a designed filter zone is placed against the impervious zone. In addition, dispersive clay can be stabilized using lime or other additives that can neutralize excess sodium salts. Use of lime requires specialized construction procedures to achieve success, which are not covered in this design standard. Industry experts, published references, and personnel with experience in the use of lime stabilization should be consulted whenever lime treatment is being considered or used.

2.2.3.4.2 Saturated Foundations

When the foundation of an earthfill dam consists of saturated fine-grained soils, the foundation soils' ability to resist the shear stresses imposed by the weight of the embankment and reservoir load must be analyzed by determining their strength and performing a stability analyses. Soils that have never been subjected to geologic loads greater than the existing overburden are "normally" consolidated. These soils are much weaker than strata that have been consolidated by hundreds or thousands of feet of ice or soil, which have since been removed. Exploration and testing to determine strength parameters are discussed in Design Standard No. 13, Chapter 12, "Foundation and Earth Materials Investigation," and the *Earth Manual* [5 and 6]. Static stability analyses requirements are discussed in Design Standard No. 13, Chapter 4, "Static Stability Analysis." Due to stability concerns, the design of the embankment is based on the results of numerous stability analyses using various interim heights of dam and different sets of slopes for the stabilizing fills for each height.

2.2.3.4.3 Relatively Dry Foundations

Since the impounded reservoir will cause the ground water to rise, unsaturated, impermeable-type soils will eventually become saturated due to construction and operation of a dam. Saturation of the foundation materials may cause a stability problem similar to that discussed previously. Effective strengths will be somewhat different because of a different consolidation history (i.e., consolidation under dry conditions versus under saturated conditions).

Additionally, some soils of low density are subject to large settlements or "collapse" when saturated by the reservoir, although these soils have high dry strength in the natural state. If proper measures are not taken to control excessive settlement, performance problems or failure of the dam may result because of: (1) differential settlement, which causes cracking (and potential internal erosion) of the impervious portion of the embankment; (2) foundation settlement, resulting in a reduction of freeboard and possible overtopping of the dam; or (3) tendency for bridging of the embankment over softer areas in the foundations and occurrence of erosive leakage (internal erosion) through the low stress areas.

These low-density soils are typified, by but not restricted to, loess, a very loose, wind deposited soil which covers vast areas of several continents, including North America. True loess has never been saturated and is generally composed of uniform, silt-sized particles bonded together with a small amount of clay. When its water content is low, loess exhibits sufficient strength to support high embankments without large settlement. A substantial increase in water content, however, greatly reduces the cohesion and may result in collapse of the loose structure of the soil under the loading imposed by even relatively low dams. Reclamation's experiences with constructing dams on loess in the Missouri River Basin are, in part, described in a paper by W.A. Clevenger [15] and Reclamation Monograph 28 [16]. Foundation consolidation is also discussed in Design Standard No. 13, Chapter 9, "Static Deformation Analysis." Davis Creek Dam in Nebraska is an example of a fine-grained loessial foundation that was completely removed from beneath the footprint of the embankment due to concerns with collapse upon reservoir wetting.

The required treatment of dry, low-unit weight foundations is dictated by the compression characteristics of the soil. These characteristics are best determined by laboratory tests on undisturbed samples at their natural water content to determine whether the postconstruction settlement caused by saturation will be significant. If the foundation is not subject to appreciable postconstruction settlement when saturated, little foundation preparation is required. The foundation should be stripped to remove organic material and proof rolled, a cutoff trench should be provided, and a toe drain should be installed to prevent saturation of the foundation at the downstream toe of the dam.

If the foundation is subject to appreciable postconstruction settlement when saturated, measures should be taken to minimize the amount of settlement. If the low-unit weight soil exists in a top stratum, it may be economical to excavate this material and replace it with compacted embankment. If the layer is too thick for economical replacement, measures should be taken to ensure that foundation consolidation is achieved during construction. Reclamation has consolidated foundations of low-unit weight loess during construction by prewetting the foundation, although a major disadvantage to this approach is the potential for large settlements during construction that may lead to embankment cracking.

2.2.3.4.4 Seismic Strength Loss

Some silt and clay foundations of low density may also be subject to loss of strength during earthquake loading. This possibility must be investigated and analyzed. As discussed in Subsection 2.2.3.3.6, "Cutoffs," extensive foundation excavation may be necessary to ensure seismic stability. Design Standard No. 13, Chapter 13, "Seismic Design and Analysis," presents methodology.

2.2.3.4.5 Seepage Control

Silts are relatively impervious; however, silts have low cohesion, and even minor seepage and low hydrostatic pressures in a silt foundation could lead to an internal erosion failure. Proper filters and drainage systems must be provided in the foundation beneath the downstream embankment section and toe area to prevent the occurrence of internal erosion.

These drainage systems are similar to those discussed for sands and gravels and should be designed in accordance with Design Standard No. 13, Chapter 5, "Protective Filters," and Chapter 8, "Seepage." Although cutoff design features for seepage control are not generally necessary in silt and clay foundations, inspection trenches through the upper portion of the foundation to cut off materials that may have been loosened by freeze thaw, roots, or desiccation and to provide inspection of the upper zone of the foundation is usually a design requirement. Additionally, there are usually requirements to inspect the foundation during construction to detect any soft or weak zones that could cause differential settlement or local movement under any part of the embankment and to remove and replace such zones. For foundations that contain large zones of weak material, it is sometimes necessary to remove and replace the material to provide stability to the embankment or to provide shear keys of higher strength material in critical zones of the foundation for stability purposes. The surface of the foundation also requires treatment during construction. These procedures are discussed in Design Standard No. 13, Chapter 3, "Foundation Surface Treatment."

2.2.4 Embankment Design for Earthfill Dams

2.2.4.1 Static Stability

Essentially, the design objective is to develop an embankment cross section and treated foundation which, when constructed with the available materials, will fulfill its required function with adequate safety at a minimum cost. Design criteria for embankments are given in Design Standard No. 13, Chapter 1, "General Design Standards." Among other requirements, these criteria require that the slopes of the embankment be stable under all conditions of construction and reservoir operation, that excessive stresses are not induced in the foundation, that seepage through the embankment and its foundation be controlled, and that the embankment be able to withstand hydrologic and seismic design loadings.

The designer of an earthfill dam cannot rely on the application of mathematical analyses or formulas to determine the required cross section to the same degree that one can for structures built of manufactured materials such as concrete. Soils occur with various combinations of particle size gradations, mineral composition, particle shapes, and corresponding variations in behavior under different conditions of saturation (moisture content), density, and loading. Further, the stress/strain relationships in an embankment are very complex. However, with advances in the field of soil mechanics, considerable progress has been made in the development of investigation, material testing, and analytical methods that will allow a comprehensive evaluation of embankment stability. These tools are particularly useful for major structures for which the cost of detailed explorations and laboratory testing of available foundation and construction materials is a small fraction of overall project costs. In these cases, a more optimum design may result from better information and analytical methods obtained at a relatively low cost. However, present practice for determining the required cross section of an earthfill dam consists largely of adopting the slopes and design characteristics of existing successful dams, taking into account the quality and quantity of materials available for construction and foundation conditions, conducting analytical and experimental studies for unusual conditions, and closely controlling the selection and placement of embankment materials.

While modifications are necessarily applied to specific designs to adapt them to particular conditions, radical innovations are generally avoided, and fundamental changes in design concepts are developed and adopted gradually through practical experience and trial. Although the practice of gradual change through verified prototype designs may be criticized as being overly conservative, no better method has been conclusively demonstrated. Where consideration is given to possible loss of life and extensive property damage that could result from dam failure, the major economic investment, and the importance of the stored water, ample justification is provided for conservative procedures.

Stability analysis of dam embankments is described in Design Standard 13, Chapter 4, “Static Stability Analysis.” The stability of an embankment depends on the driving forces provided by gravity (and sometimes by earthquake loading), the strength properties of embankment and foundation materials, and the pore-water pressure within those materials. The principles and procedures are well established in practice, although unusual situations may arise that require deviation from standard practice.

Material strength properties are governed by the nature of the soil (type, gradation, and mineralogy, especially with clays) and the density, whether in embankment or foundation. In general, coarser soils and more angular soils have higher shearing resistance. Minimum density requirements are imposed to ensure that the fill has adequate strength. Foundation soils may require removal or densification if they are determined to be too weak or too prone to settlement.

The shearing resistance increases with the effective normal stress, equal to the imposed normal stress minus the pore-water pressure. Hence, the embankment designs generally include zoning to minimize pressure within and below the downstream slope, so that the slope can be made steeper and require less fill. In order to maintain stability of the upstream slope during rapid lowering of the reservoir, it may be necessary to provide free-draining material within the upstream slope to ensure that high pore pressure does not remain within the embankment.

2.2.4.2 Seepage and Leakage through Embankments

The core or water-barrier portion of an earthfill dam provides the resistance to seepage and creates the reservoir. However, soils vary greatly in permeability, and even the tightest clays have some permeability and cannot prevent water from seeping through them. In addition, cracks in the water barrier can be caused by differential settlement, desiccation, frost action, hydraulic fracturing, etc. Paths of seepage or leakage can also be caused by construction deficiencies such as poorly compacted lifts or placement of coarse, pervious lifts from a variable borrow area.

The progress of percolation of reservoir water through the core depends on the consistency of the reservoir level, the permeability of the core material in the horizontal and vertical directions, the magnitude of residual pore-water pressures caused by compressive forces during construction, and the element of time. The upper surface of the seepage zone is called the phreatic (zero pressure) surface; in cross section, it is referred to as the phreatic line. See the discussion in Subsection 2.2.2.1.2, "Homogeneous Embankments."

Although the soil may be saturated by capillarity above the phreatic line, giving rise to a "line of saturation," seepage is limited to the portion below the phreatic line.

The position of the phreatic line depends on the reservoir level, geometry of the embankment section, ratio of core permeability to shell permeabilities, and ratio of horizontal to vertical permeability. For embankments constructed of soils of vastly different permeabilities, but of the same ratio of horizontal to vertical permeability in a homogeneous dam, the phreatic lines eventually will reach an identical position. It will take much longer for the steady-state condition to be reached in clay than in sand for the same embankment cross section, and the amount of water emerging at the downstream slope will be much greater in the more pervious material.

Methods of controlling seepage and leakage through the embankment are discussed in Subsection 2.2.2, "General Comments on Earthfill Dams." Design and analysis requirements for these features are presented in Design Standard No. 13, Chapter 5, "Protective Filters," and Chapter 8, "Seepage."

In addition to seepage that occurs through the embankment proper, special attention should be given to seepage or leakage that occurs through an embankment in the vicinity of any structure penetrating the embankment such as an outlet conduit or spillway. The earthfill in the vicinity of a rigid structure and the earth structure interface are areas where uncontrolled seepage and potential internal erosion can develop for several reasons:

- Placement and compaction of earthfill are more difficult using motorized equipment adjacent to the structure. In the past, this usually has resulted in the use of hand-operated placement and compaction equipment near the structure.

- A hand-placing and compaction operation is labor intensive and less effective than a motorized spreading and compacting operation.

- The interface zone between the two operations is a troublesome zone. Motorized equipment operators naturally avoid labor crews on foot. Hand compaction tends to lag behind motorized equipment compaction, causing unequal fill surface heights. As a result, the interface zone between the two operations often receives inadequate compaction.

- Because hand compaction is slow, tends to lag behind compaction using large equipment, requires more effort to obtain proper moisture and density, may require special gradation of soil particles, and requires intense inspection, it is a source of irritation for both the contractor and the owner. This results in a tendency to concentrate more on progress of the earthfill placement rather than good construction techniques.

- Hand compaction requires thinner lifts and more time to achieve the specified compaction, and scarification of lift surfaces is difficult. These factors increase the probability of poorly bonded lift surfaces that may develop into seepage paths and areas that could be jacked apart (hydraulically fractured) by water pressure.

- Stress distribution around the structure is nonuniform with a tendency for earth pressure to arch onto the structure, causing low stresses within the earthfill and a greater opportunity for the occurrence of hydraulic fracturing through the impervious zones. Irregular structure surfaces complicate this problem even further.

These problems have slowly led to generally accepted practices among earth dam designers. Structures through embankments should be avoided unless economics or site geology dictate their use. If they are used, the primary means of controlling seepage or leakage along the surface of the structure, or through adjacent impervious zones, is the use of a properly designed filter and drainage zones around the conduit downstream of the impervious core, along with quality constructed fill adjacent to the structure.

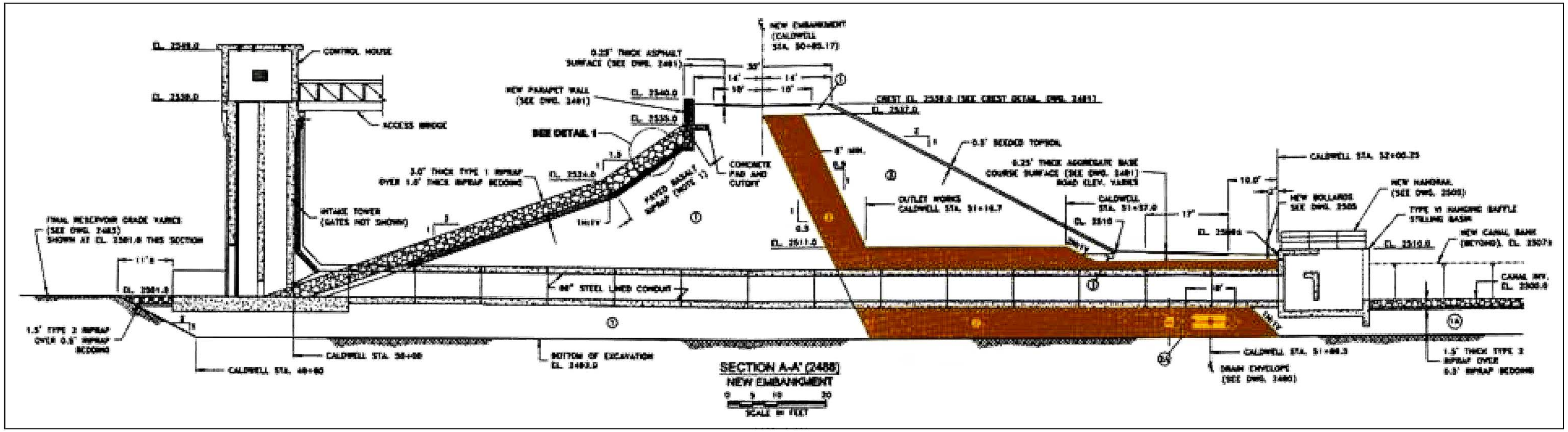
Previously, Reclamation has used cutoff collars around conduits in the section of the conduit through the impervious zones of embankment dams to help control seepage. There have been no documented problems with Reclamation dams as a result of this practice. However, the majority of embankment dam engineers argue that cutoff collars do not perform the intended purpose of controlling seepage and could be detrimental. Compaction of the embankment around cutoff collars has the same problems as discussed previously for rigid structures through the embankment. The pros and cons of cutoff collars are discussed in Assistant Commissioner – Engineering and Research (ACER) Technical Memorandum No. 9, “Guidelines for Controlling Seepage Along Conduits Through Embankments,” [17] which was prepared by a task group of Reclamation engineers. An additional excellent reference is a technical manual on conduits through embankments sponsored by the Federal Emergency Management Agency (FEMA) [18]. Reclamation policy is that cutoff collars should not be used as a seepage control measure, and any other protruding features on a conduit should be avoided.

The filter/drainage system should completely surround the conduit in the area immediately downstream of the impervious core where the conduit is founded on soil. Figure 2.2.4.2-1 shows examples taken from Design Standard 13, Chapter 5, "Protective Filters." Similar types of conduit envelopes should be considered at both new and existing embankment dams. If the conduit is founded on rock, consideration can be given to only surrounding that portion of the conduit that is within embankment fill, depending on the competency of the rock.

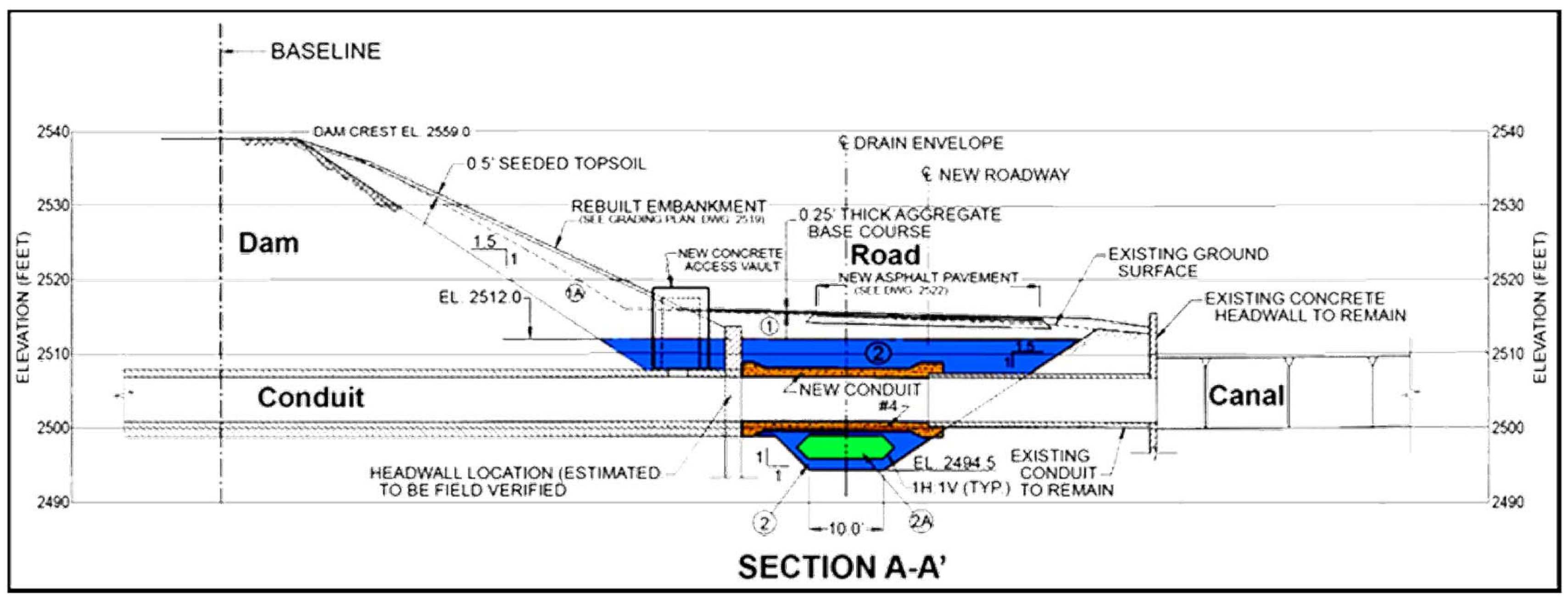
The chimney filter/drain can normally be used to fulfill this requirement. Additionally, provision must be made to convey any seepage or leakage collected safely out of the interior of the embankment. This can usually be accomplished by abutting the horizontal filter/drainage blanket against the concrete structure. This portion of the filter/drainage system does not necessarily envelop the structure or conduit, but it must have adequate hydraulic capacity and filtering characteristics and must be connected to the protective filter/drain around the conduit. If the internal filter/drainage system cannot be combined to provide adequate filtering and drainage for structures through the embankment, a separate filter/drainage system should be designed for the structure. Refer to Design Standard No. 13, Chapter 5, "Protective Filters," and Chapter 8, "Seepage," for details on filter/drain design.

To facilitate a high-quality constructed fill through the impervious core adjacent to the conduit or structure, surfaces of the structure should be smooth and vertical surfaces should have a minimum batter of 1:10 H:V. Motorized compaction equipment should be used to the greatest extent possible to compact fill adjacent to and against the structure. This can be accomplished and facilitated by ramping the fill slightly, 6:1 H:V slope, near the structure and operating pneumatic-tired, motorized equipment parallel to the structure face or wall. Figure 2.2.4.2-2 (taken from Design Standard No. 13, Chapter 5, "Protective Filters," demonstrates these design/construction details. Ideally, impervious earth material that is placed adjacent to a conduit would be reasonably well graded; have a maximum particle size smaller than 1 inch (including earth clods); a minimum of 50 percent, by weight, passing a No. 200 sieve; and a plasticity index between 15 and 30.

Earthfill should be dumped and spread in lifts parallel to the structure in such a manner to ensure that these layers of material that extend along the structure are of a nature and permeability consistent with the adjacent earthfill in the embankment. The fill should be compacted using pneumatic-tired rollers or equipment with wheels operated against the ramped fill surface immediately adjacent and parallel to the structure. The lift should be compacted to 6 inches or less, and the surface should be scarified before placement of the next lift. Moisture content during compaction should be at or slightly wet of optimum, and the compacted dry unit weight should be equivalent to that required in normally compacted embankment not affected by structures.



(a)



(b)

Figure 2.2.4.2-1. Examples of filters around penetrating conduits: (a) typical filter addition around a conduit near the centerline of a dam, and (b) typical filter addition around a conduit near the downstream toe of a dam

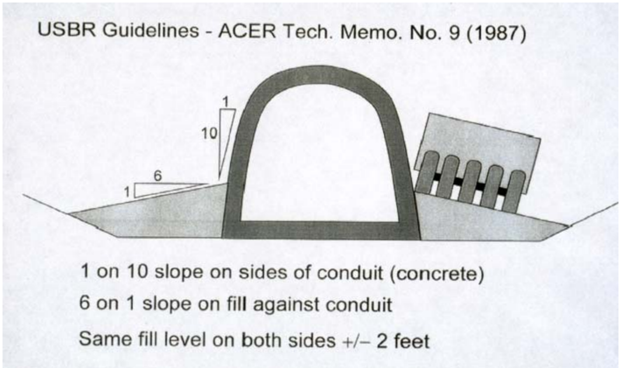


Figure 2.2.4.2-2. Recommended design and compaction details for penetrating conduit.

2.2.4.3 Utilization of Materials from Required Excavation

In Subsection 2.2.2.3, "Criteria for Design", it was pointed out that economic considerations require that the dam be designed to make maximum use of the most economical materials available, including material which must be excavated for the dam foundation, spillway, outlet works, canals, powerhouses, roadways, and other appurtenant structures. When the yardage from these sources constitutes an appreciable portion of the total embankment quantity, the availability of these materials may strongly influence the design of the dam. Even if these materials may be less desirable than soil from available borrow areas, economy may dictate that they be considered. Both available borrow areas and structural excavations should be considered when developing a suitable design. Materials from structural excavation require exploration and laboratory testing programs equivalent to those used to evaluate borrow area materials to determine their adequacy, appropriate zone of use, and available volume. Material from required excavation may have to be stockpiled for later use in the embankment. More savings can be realized, however, if scheduling the construction of various features allows direct use of required excavation.

The portion of the cutoff trench excavation that is above the ground water table may provide material for the impervious core of the dam or may provide sand and gravel for filters, drains, and shells. Sand and gravel may also be available in the dewatered portion of the trench from the strata that are being intercepted by the cutoff trench excavation. When sand and gravel occur in thick, clean beds, this material can be used in the outer zones of the dam or processed to obtain filter and drain materials. However, pockets or lenses of silt and clay, as well as highly organic material, sometimes occur in cutoff trench excavations. These materials can contaminate the clean soils and may result in overly wet mixtures of fill material having variable permeability and poor workability if proper control is not exercised during construction. These soil mixtures may be used in miscellaneous or random fill zones or may have to be wasted.

Excavation for the spillway may provide both overburden soils and formation rock. In planning the use of these materials, the designer must recognize that stockpiling, moisture control, processing, and special size requirements may add to the project cost. For these reasons, material from spillway excavations is primarily used in the main structural zones (shells) of dam embankments.

Although small in volume, tunnel excavations can also provide rockfill material for use in transition or pervious zones of the dam.

The feasibility of using materials from structural excavations is influenced by the sequence of construction operations. The construction sequence is, in turn, influenced by the following items:

- Topography of the damsite
- Diversion requirements
- Hydrology of the watershed
- Seasonal climatic changes
- Magnitude of required excavations
- Contractor operations

The placement of material directly from the spillway or cutoff trench excavation on the embankment, without having to stockpile and rehandle it, requires providing an adequate placing area. The placing area is usually restricted early in the job; hence, the designer is faced with the choice of specifying that spillway excavation be delayed until space is available for placement, requiring extensive stockpiling, or permitting large quantities of material to be wasted. The amount of embankment space that can be provided during the early stages of construction depends, in part, on the contractor’s operations including diversion requirements and plan. Usually, the contractor is allowed considerable flexibility in overall operations and the method of diversion, which adds to the designer's uncertainty in planning the use of materials from structural excavations.

Zoned dams provide an opportunity to specify the use of materials from structural excavation. The zoning of the embankment should be based on the most economical use of materials; however, the zoning must be consistent with the requirements for stability and seepage control, as previously discussed. For example, the use of rockfill sections can allow continuing construction during wet or cold weather conditions, thus extending the construction season.

An important use of materials from structural excavation is in portions of the embankment where the permeability and shear strength are not critical and where weight and bulk are the major requirements. Stabilizing fills required for dams on weak foundations are an illustration of this usage.

Areas within the dam into which such excavated material is placed are called “random zones.” Figure 2.2.4.3-1 shows typical locations for these random zones.

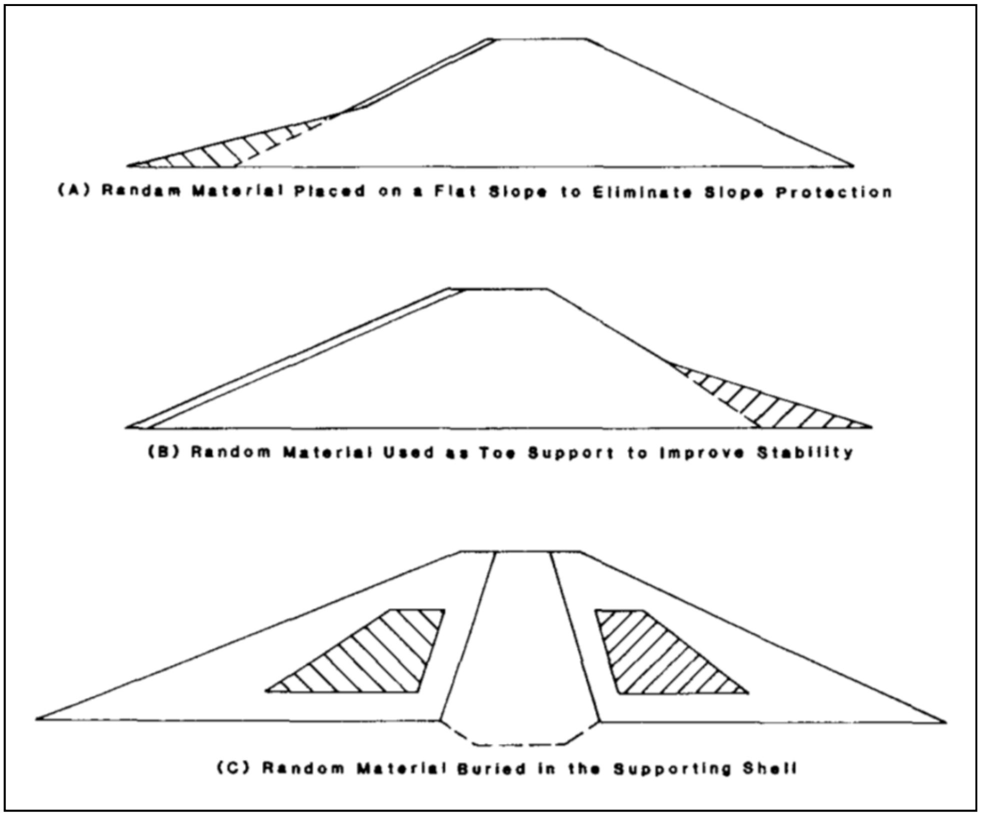


Figure 2.2.4.3-1. Use of random fill materials within an embankment.

Because estimates of the percentage of structural excavations usable within the embankment are subject to variation, variable zone boundaries (such as utilized at Anderson Ranch Dam) to accommodate any excess or deficiency are sometimes required. In some cases, special laboratory tests or a test embankment may be required before determining the disposition of questionable material or selecting the dimensions of a random zone. In formulating a design, the designer must anticipate what percentage of the structural excavation will be suitable for the various zones of the embankment and the yield factors (shrinkage and swell) of the material involved. The designer must then integrate these anticipated quantities with the required borrow area quantities to determine a final design which is both economical and has a reasonable construction sequence. Typical shrinkage and swell factors for various materials can be found in the literature; one example is the 1981 *Excavation Handbook*, authored by Horace K. Church.

Often, several zoning schemes are considered in order to optimize the use of required excavation materials. The use of a materials distribution chart, such as shown in figure 2.2.4.3-2, has been helpful for integrating excavation quantities into the embankment section, for determining the required amounts of borrow material for each zone, and for visualizing the construction sequence. The chart shown is for Reclamation's New Waddell Dam in Arizona. In addition to showing the sources of all fill materials, the chart contains the assumed shrinkage, swell, and yield factors on which specifications quantities are based.

2.2.4.4 Zoning

2.2.4.4.1 General

The design slopes of an embankment may vary widely depending on the character of the materials available for construction, foundation conditions, and the height of the structure. Pervious foundations may require the addition of impervious upstream blankets to reduce the amount of seepage or the addition of downstream horizontal drainage blankets to provide stability against seepage-induced uplift forces. Weak foundations may require the addition of stabilizing fills at either or both toes of the dam.

The slopes of an earthfill dam depend on the type of dam (i.e., diaphragm, modified homogeneous, or zoned embankment) and on the nature of the materials available for construction. Of special importance is the nature of the soil that will be used for construction of the modified homogeneous dam or the core of a zoned dam. For a zoned embankment, the width of the core will also have a significant impact on the outer slopes; in general, the wider the core, the flatter the outer slopes.

Embankment slopes are generally estimated during the early stage of design on the basis of experience with previous construction materials and foundations, then verified by stability analyses and adjusted as necessary during final design. The initial estimate should include appropriate contingencies to ensure that cost estimates are adequate.

Flat upstream slopes are sometimes used in order to eliminate or reduce expensive slope protection. A berm can be provided at an elevation about 5 feet below the inactive capacity water surface elevation to form a base for the upstream slope protection, which does not need to be carried below this point. The upstream slope is often steepened above the elevation where water is stored (i.e., in the surcharge range). The slope in the range of any joint-use or flood control storage should be considered on a case-by-case basis.

The rate of reservoir drawdown is an important factor that affects the stability of the upstream slope of the dam. If the reservoir drawdown occurs at a fast enough rate, pore pressures in the upstream zone and foundations of the embankment may not have time to dissipate. This may result in failure of the upstream slope. A storage dam subject to rapid drawdown of the reservoir should have an upstream zone of sufficient size and permeability to dissipate pore-water pressures during drawdown. A method for designing free-draining upstream shells is shown in Cedergren [20], page 148. Where only fine material of low permeability is available, it may be necessary to provide a flat slope for rapid drawdown stability. If free-draining sand and gravel, or sound and durable rock, are available for construction of the upstream zone, a steeper slope may be used provided that the foundation has adequate strength. If rockfill is used, a transition layer of sand and gravel between the rockfill and the surface of the impervious embankment may be necessary to prevent fine-grained material from migrating into the rockfill.

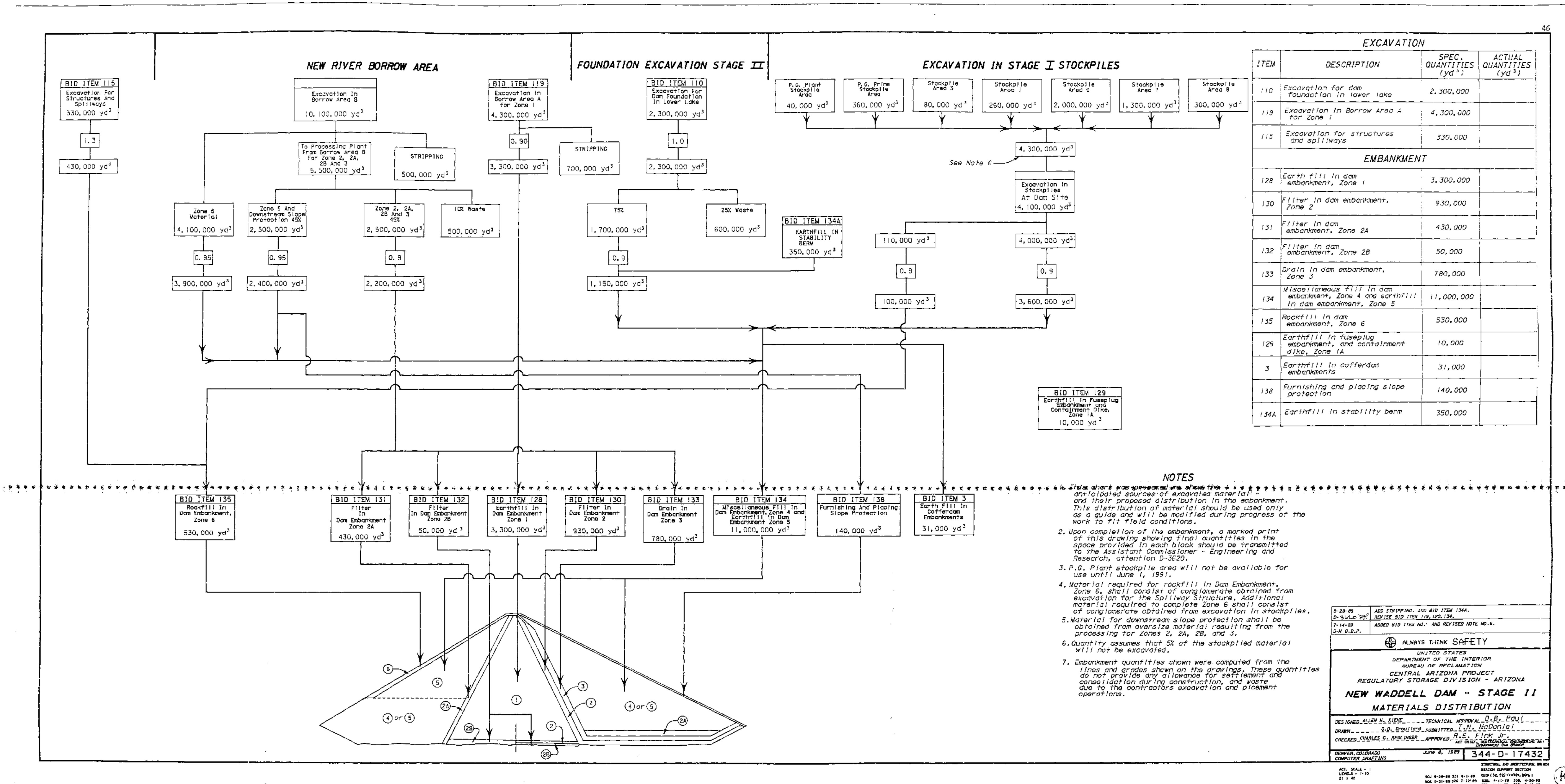


Figure 2.2.4.3-2. Example materials distribution chart

The reservoir water load and hydrostatic pressures act as a stabilizing influence on the upstream zone of an embankment when the reservoir is full. In the absence of an unusual loading condition, such as an earthquake, an upstream failure would generally only be possible during construction or following a rapid drawdown; in both cases, the reservoir should be virtually empty. Therefore, loss of the reservoir due to failure of the upstream slope under these static loading conditions is very unlikely. However, the dam could be out of service for a long period of time and require costly repairs or replacement, resulting in economic impact to the water users and the local community.

2.2.4.4.2 Diaphragms

Diaphragm-type dams are discussed in Subsection 2.2.2.1.1, “Diaphragm Embankments,” and are generally used under the following conditions:

- A limited quantity of impervious material is available
- Wet climatic conditions
- Short construction seasons

The pervious material used in the construction of a diaphragm-type dam must be capable of compaction to form a stable embankment which will be subject to only small amounts of postconstruction settlement. However, if the membrane is an interior earth diaphragm that is centrally located, the designer must consider the potential that low stresses may develop within the diaphragm if surrounding transitions and filters are too stiff to consolidate with the diaphragm. This condition could cause fracturing or cracking of the diaphragm. Poorly graded sands (SP) are difficult to compact but can serve as an embankment. Warren H. Brock Reservoir in California is an example of a recent Reclamation facility that has an upstream geomembrane serving as the diaphragm in an otherwise homogeneous dam constructed from poorly graded sands. If available, well-graded sand-gravel mixtures (SW or GW) or well-graded gravels (GW) provide more satisfactory embankments. Well-graded sand-gravel mixtures that contain more than 5 percent passing the No. 200 sieve should be tested to confirm that they will be relatively free-draining after compaction. Normally, the permeabilities of well-graded sand and gravel soils are more sensitive to the percent fines (minus No. 200 sieve) than poorly graded sand and gravel soils.

Except for the use of pervious materials other than rock in construction of the embankment, the diaphragm earthfill dam design is similar to the design of rockfill dams, which is discussed in Section 2.3, “Rockfill Dams.” That discussion should be referred to for the design of foundations and upstream facings for a diaphragm-type earthfill dam.

2.2.4.4.3 Homogeneous Dams

As previously discussed, purely homogeneous dams are not recommended for new dams. Only modified homogeneous dams which provide for the inclusion of a chimney filter/drain and drainage blanket are recommended. Such a dam may be considered a special case of a zoned embankment. Internal drainage should be designed in accordance with Design Standard No. 13, Chapter 5, “Protective Filters,” and Chapter 8, “Seepage.”

2.2.4.4.4 Zoned Embankments

Zoned embankments are discussed and examples are shown in Design Standard No. 13. Chapter 1, “General Design Standards.”

The zoned embankment dam has led to more economical structures where there are a variety of soils readily available. Three major advantages in using zoned embankments are:

- Steeper slopes may be used, with consequent reduction in total volume of embankment material and shorter length of appurtenant structures.

- A wide variety of materials may be used.

- Maximum use can be made of material excavated from the foundation, spillway, outlet works, and other appurtenant structures.

Zoning schemes are based on the estimated quantities of required excavation and borrow materials available. The scheme of zoning may divide the dam into two, three, or more zones, depending on the variation of the engineering properties of the available materials for construction. A filter on the downstream side of the impervious core should always be provided and should be designed in accordance with Design Standard No. 13, Chapter 5, “Protective Filters,” and Chapter 8, “Seepage.” In general, the permeability of each zone should increase toward the outer slopes. Relatively free-draining materials and, therefore, those with a high degree of inherent stability are used to support the less stable impervious core and filter. Pervious materials, if available, are generally placed in upstream zones to permit the dissipation of pore pressure during rapid drawdown. If pervious materials are not available, naturally occurring materials in the area may be used, but a flatter slope may be necessary for adequate stability during rapid drawdown.

Miscellaneous or random zones, as shown on figure 2.2.4.3-1, are often included in the downstream zones of the embankment to use excavated materials of uncertain permeability. Excavated materials not suited for use in any zone and excess excavation may be wasted on the upstream or downstream toes; however, care must be taken not to damage downstream toe drains or bury them to the point of inaccessibility. Subsection 2.2.4.3, “Utilization of Materials from Required Excavation,” discusses more fully the use of excavated material.

2.2.4.4.5 Transition Zones

It is important that the gradation of adjacent zones be considered so that materials from one zone are not eroded into the voids of adjoining zones, either by steady-state seepage or by rapid drawdown seepage forces. Transition zones (and filters) protect against internal erosion and provide the additional advantage that, should the embankment crack, partial self-healing of the cracks takes place with subsequent reduction in seepage losses. Filters and drains designed in accordance with Design Standard No. 13, Chapter 5, "Protective Filters," should be provided downstream of the impervious core.

Another purpose of transition zones is to reduce the amount of cracking that would be caused by differential deformation if two embankment zones with greatly differing deformation modulus are placed immediately adjacent to each other. For example, if a centrally located clay core is placed between shells of well-graded, densely compacted gravel, the core may tend to consolidate more than the gravel shells. Shear forces will develop at both faces of the core that tend to prevent its consolidation and cause cracking within the core. A transition zone consisting of an intermediate modulus material is required between the core and the shells. This transition zone usually consists of a granular material that is 10 to 12 feet wide and is compacted to a density less than that of the shells. The filter downstream of the core can usually serve the dual purpose of a filter and a transition modulus zone. A specific zone may be required upstream of the core, but it may also serve as a "crack stopper" to supply cohesionless material that may help to self-heal a crack that occurs in the impervious core.

2.2.4.5 Seismic Design

Embankment dams must be designed to withstand earthquake loading without catastrophic release of the impounded reservoir. Potential for seismic loading should be considered for all embankment dams, and the dam must be designed to withstand the seismic loading as necessary. To mitigate the potential for seismic loading to cause damage to the embankment, the designer may consider the excavation or treatment of liquefiable materials from beneath the shells of the embankment and the construction of a stability berm at the downstream toe. Procedures for investigating seismic stability are given in of Design Standard No. 13, Chapter 13, "Seismic Design and Analysis."

2.2.4.6 Security Considerations

The design of embankment dams must consider the impacts of a potential security-related incident, such as an explosive device placed on the crest. The potential for catastrophic failure from a blast loading depends on a variety of factors including embankment type, materials, crest width, freeboard, physical access to the crest, and amount and type of explosives. Staff in the Security Office and Technical Service Center can provide data on the predicted impacts of different blast loads. This allows the designer to determine if the proposed embankment design is sufficient to protect the structure for different design-based threats. The Reclamation Security Office should be contacted for additional information and consultation.

2.2.5 Embankment Details for Earthfill Dams

2.2.5.1 Crest Details

2.2.5.1.1 General

When designing the crest of an earthfill dam, the following items should be considered:

- Width
- Drainage
- Camber
- Surfacing
- Public safety
- Zoning

Constructability often controls the crest details. As the embankment construction reaches the crest of the dam, the working area becomes extremely limited for hauling, placing, and compaction equipment. Designers usually select crest details, which may involve narrowing zones near the crest that will accommodate construction operations.

It is usually desirable to prohibit public access to the crest of the dam because of vandalism and security issues. Public parking for visitors and recreational users should be located at a separate location away from the dam structure. Where the dam crest road dead ends at an abutment, a turnaround should be provided for maintenance vehicles. Parking for operation and maintenance vehicles should also be provided at gate or instrument houses on the dam crest. Fencing, locks, etc., should be provided as appropriate, both to protect the public and to prevent vandalism.

2.2.5.1.2 Width

The crest width of an earthfill dam depends on considerations such as: (1) properties of embankment materials and minimum allowable seepage distance through the embankment at normal reservoir water level, (2) roadway requirements, (3) practicability of construction, (4) designs for dams in high seismic areas, (5) any planned future crest raises, and (6) potential security-related vulnerabilities. A minimum crest width should provide a reasonably low seepage gradient through the embankment at the level of the maximum reservoir. In highly seismic zones, a wider crest provides greater safety against a breach of the dam during a large magnitude earthquake. An increase in crest width will generally result in a reduction in security-related vulnerabilities.

2.2.5.1.3 Drainage

Surface drainage of the crest should be provided by a crown with a 2-percent slope to the edges or by sloping the crest at a 2-percent slope to drain towards the upstream slope. The latter method is preferred unless the downstream slope is protected against erosion by some type of slope protection or if environmental considerations dictate otherwise.

2.2.5.1.4 Camber

Camber is ordinarily provided along the crest of earthfill dams to ensure that the freeboard will not be diminished by postconstruction foundation consolidation and embankment compression. Selection of the amount of camber is based on the amount of foundation consolidation and embankment compression expected, with the objective of ensuring that the crest elevation remains at or above the design crest elevation after settlement. Camber also improves the aesthetic appearance of the crest from a distance. The top of the impervious zone should also be cambered similarly to that provided for the crest of the dam so that it does not settle below the maximum water surface elevation (see figure 2.2.5.1.4-1).

Impervious embankment materials placed at densities roughly corresponding to the standard Proctor laboratory maximum will consolidate appreciably when subject to overlying fill loads. It is expected that the major portion of this consolidation will take place during construction before the embankment is completed. For dams on relatively incompressible foundations, cambers of about 1 percent of the height are commonly provided. Consolidation of a compressible foundation, in which drainage is slow, may be a more important factor in estimating camber than embankment settlement.

Several feet (1 to 2 percent of dam height) of additional camber may be required for dams constructed on foundations which may be expected to settle. Note, however, that dams founded on soils that would be expected to settle appreciably have other design issues, such as the potential for cracking, that need to be considered. Methods of determining foundation settlement are given in Design Standard No. 13, Chapter 9, “Static Deformation Analysis,” as is camber design. Straight-line variation should be used to describe the amount of camber and to make it roughly proportional to the height of the embankment above its foundation. This method is easy to use in the office and easy to construct. Figure 2.2.5.1.4-1 shows an example of camber designed for Jordanelle Dam.

The additional amount of embankment material required to provide camber is usually nominal, and the increased height of the embankment is provided by oversteepening the slopes near the crest of the dam, as shown in figure 2.2.5.1.4.-1. The modifications to the zones of the embankment, due to the addition of camber, are not considered in calculating embankment stability.

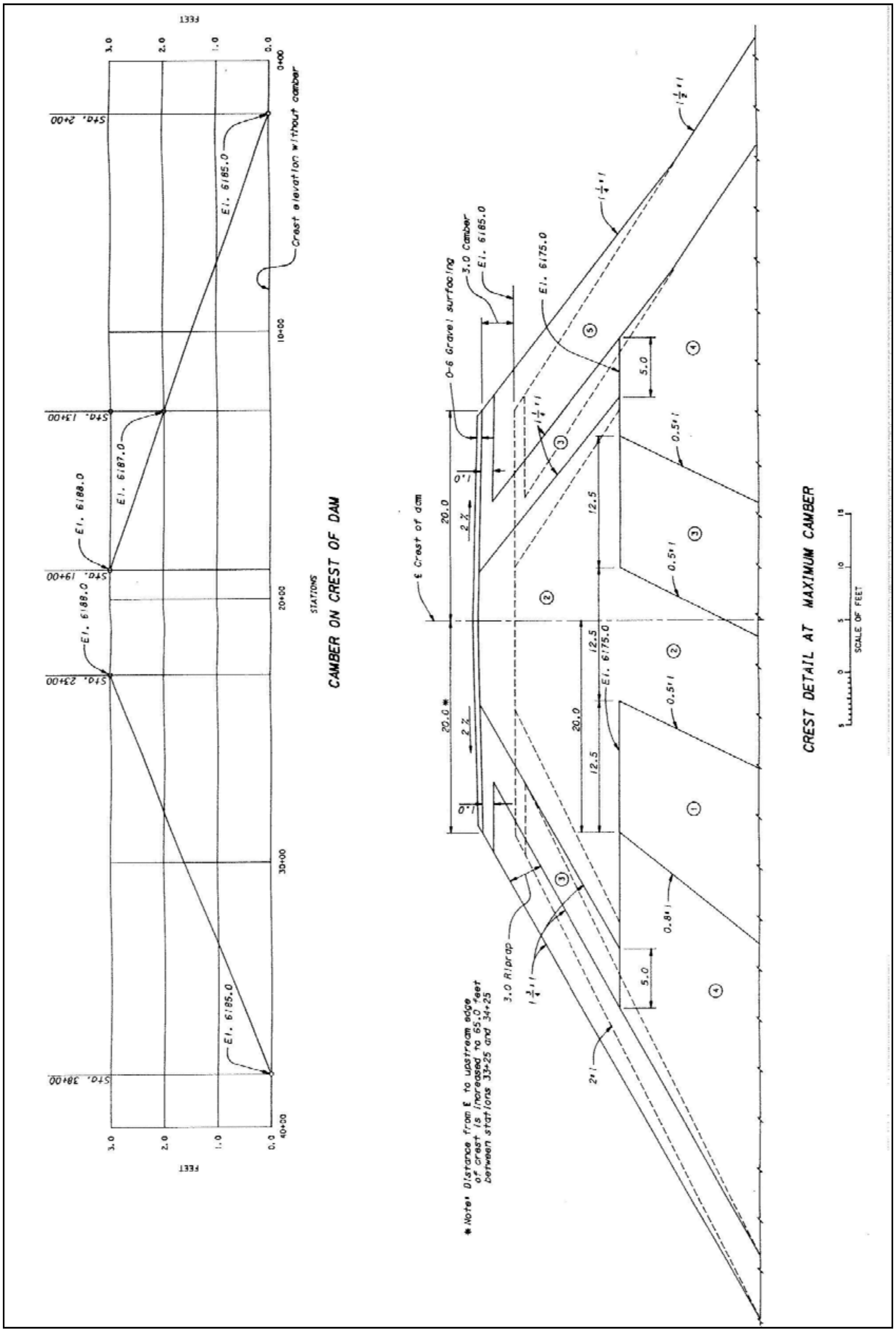


Figure 2.2.5.1.4-1. Crest and camber details - Jordanelle Dam, Utah.

2.2.5.1.5 Surfacing

Some type of surfacing should be placed on the embankment crest to provide protection against damage by wave splash and spray, rainfall runoff, wind, frost action, and traffic wear when the crest is used as a public roadway. The usual treatment consists of placing a layer of selected fine rock or gravelly material to a minimum thickness of 6 inches. If a public roadway traverses the crest of the dam, the width of the roadway and type of surfacing should conform to connecting public roadway requirements. Even where paving is not a traffic wear consideration, it is advantageous to have the crest paved for protection from wave splash, spray, and runoff. Paving will also provide some protection against overtopping, even though this should not be considered during design. If cracking occurs, it will likely be manifested through the paving and be noticed, whereas it may go unnoticed in gravel surfacing or no surfacing.

2.2.5.1.6 Public Safety

If the crest of the dam is to be used as a public roadway, beam-type guardrails are usually constructed along both shoulders of the crest. If a highway crossing is not anticipated, the crest can be delineated by posts at 25-foot intervals or, on very minor structures, by boulders placed at intervals along the crest, although, in many instances, no treatment is required. Existing safety standards should be reviewed, however, to ensure conformance.

2.2.5.1.7 Zoning

Poor zoning design at the crest leads to poor construction control, lost time, and, possibly, local failure of the crest. In both homogeneous and zoned dams, considerations must be given to the manner in which the slope protection and bedding will intersect the crest. The thickness of the slope protection may have to be reduced by steepening the slopes near the crest to allow construction of the impervious or pervious zones or to facilitate the installation of guardrail posts. Care must be taken to ensure that the remaining slope protection will adequately resist wave action. The oversteepened slopes, provided for camber, should avoid being overly steep to facilitate construction.

In homogeneous or modified homogeneous dams, where shrinkage cracks or frost action may be problems, crest surfacing with asphaltic concrete or concrete is desirable. In zoned dams, it is common practice to limit the height of the core material to a few feet below the crest because impervious zones extending to the top of the dam are subject to damage by desiccation and frost action, which cause loosening and cracking of the soil. Zoning around the top of the impervious core, and additional core height above the maximum water surface, should be provided to control seepage through the embankment. Following are some design features that should be considered to ensure adequate performance in the upper portion of the dam:

- Top of the impervious core should be at or above the maximum reservoir water surface anticipated during the inflow design flood.
- Chimney filter should extend to the top of the impervious core.
- Drainage zone should extend high enough to reliably collect seepage in the upper part of the dam.
- Sufficient cover should be provided over the impervious core to protect it from freezing or desiccation.

It is not unusual for longitudinal cracks caused by differential settlement between zones to develop in the crest of a dam. The degree of cracking can be reduced by providing transition zones between materials with large differences in deformation modulus. This is sometimes done by reducing the compaction effort in transition or filter zones so that a transition of modulus is obtained from one material to the other. Care must be maintained, however, to ensure adequate compaction in the transition zone for strength and to preclude internal erosion.

2.2.5.1.8 Typical Crest Details

Figure 2.2.5.1.4-1 shows the crest detail for Jordanelle Dam; 3 feet of camber was provided across the maximum section area, and a minimum top width of 12.5 feet was maintained for the impervious zone and filters/drains to ensure adequate room for placement and compaction. Figure 2.2.5.1.8-1 shows additional crest details for various Reclamation dams.

2.2.5.2 Freeboard

Freeboard is the vertical distance between the crest of the embankment (without camber) and the reservoir water surface. The more specific term “normal freeboard” is defined as the difference in elevation between the crest of the dam and the top of active conservation, joint use, or exclusive flood control water level as fixed by design requirements. The term “minimum freeboard” is defined as the difference in elevation between the crest of the dam and the maximum reservoir water surface that would result from a routing of the design flood with the outlet works and spillway operating as planned. Some allowances should be made for malfunction of the spillway and outlet works gates and for security-related incidents where the possibility exists. This is called “robustness” of design and is discussed further in Design Standard No. 14, Chapter 2, “Hydrologic Considerations.” The difference between normal and minimum freeboard represents the surcharge head. If the spillway is uncontrolled, the design will always include a surcharge head; if the spillway is gated, it is possible for the normal and minimum freeboards to be similar. It is also possible for the normal freeboard requirement to control the elevation of the crest of the dam because of the greater probability of higher waves during normal use. Freeboard requirements and methods for determining freeboard are discussed in detail in Design Standard No. 13, Chapter 6, “Freeboard.”

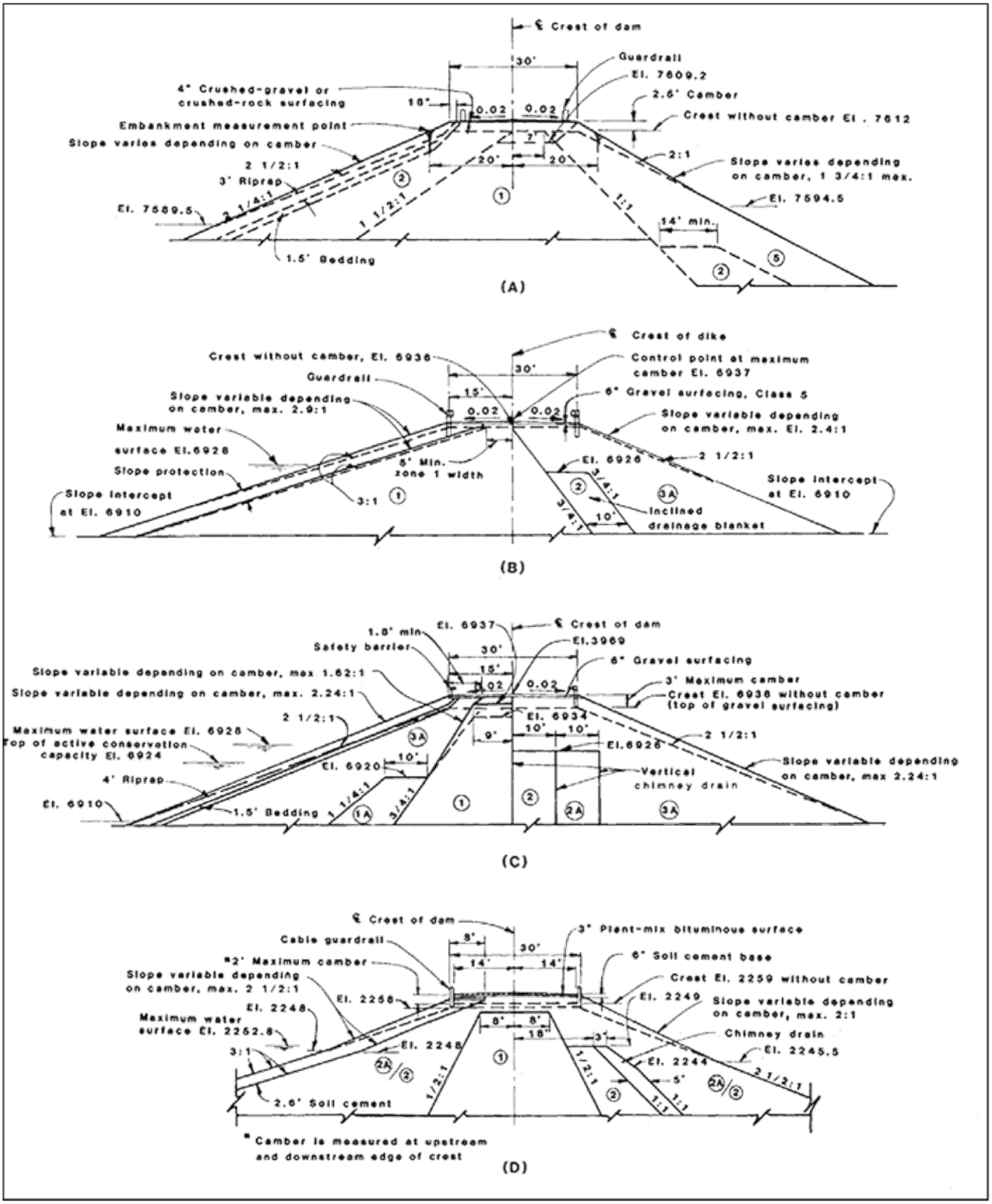


Figure 2.2.5.1.8-1. Examples of crest details at maximum camber.

2.2.5.3 Upstream Slope Protection

2.2.5.3.1 General

The upstream slopes of earthfill dams must be protected against destructive wave action. In some instances, provision must be made against burrowing animals (e.g., an upstream diaphragm or impervious zone). Usual types of surface protection for the upstream slope are dumped rock, riprap, and soil-cement. Other types of protection that have been used are steel facing, concrete pavement, asphaltic concrete pavement, precast concrete blocks, and (on small and relatively unimportant structures) wood and sacked concrete. The upstream slope protection normally extends from the crest of the dam to a safe distance below the top of the inactive capacity water surface (usually about 5 feet). In some cases, it is advantageous to terminate the slope protection on a supporting berm. Because of the high cost of upstream slope protection, consideration may be given to only providing bands of riprap in zones of more frequent reservoir surfaces and at the crest of the dam. In this case, protection against surface runoff would have to be provided in areas without riprap.

2.2.5.3.2 Selection of Type of Slope Protection

Experience has shown that, in the majority of cases, properly graded and placed riprap with adequate durability properties furnishes the best type of upstream slope protection at the lowest cost. Reclamation experience with riprap is summarized in Dams Branch Report No. DD3, “Rock as Upstream Slope Protection for Earth Dams - 149 Case Histories” [21] and REC-ERC-73-4, “Riprap Slope Protection for Earth Dams: A Review of Practices and Procedures” [22]. Approximately 100 dams, located in various sections of the United States with a wide variety of climatic conditions and wave severity, were examined by the U.S. Army Corps of Engineers to provide a basis for establishing the most practical and economical means for slope protection [23]. The dams ranged in age from 5 to 50 years old and were constructed by various agencies. This survey found that:

- Dumped riprap failed in 5 percent of the cases where it was used; failures were attributed to improper size of stones.
- Hand-placed riprap failed in 30 percent of the cases where it was used; failures were attributed to the lack of interlocking resulting from single-course construction.
- Concrete pavement failed in 36 percent of the cases where it was used; failures were due to poor design and construction details.

This survey substantiated the premise that dumped riprap, described in the next section, is the preferable type of upstream slope protection.

2.2.5.3.3 Dumped Rock Riprap

Dumped rock riprap consists of a reasonably well-graded distribution of stones or rock fragments that are dumped in place on the upstream slope of an embankment to protect it from wave action. The riprap is placed on a properly graded filter (bedding) which may be a specially placed blanket layer or may be the upstream zone of a zoned embankment and is filter-compatible with adjacent zones. Riprap design is discussed in detail in Design Standard No. 13, Chapter 7, “Riprap Slope Protection.”

The excellent service rendered by dumped riprap is typified in the case of Horsetooth Dam, constructed by Reclamation. Figure 2.2.5.3.3-1 shows the excellent condition of the riprap on the upstream slope of this dam after more than 60 years of service.

Figure 2.2.5.3.3-1. Riprap on upstream slope of Horsetooth Dam, Colorado.

The superiority of dumped rock riprap for upstream slope protection and its low cost of maintenance, when compared to other types of slope protection, have been demonstrated so convincingly that it has been considered economical to import rock from considerable distances to avoid construction of other types of slope protection for major dams. For example, Reclamation has imported rock from sources which required a rail haul of over 200 miles and a truck haul of 24 miles from the railhead to the dam, and the U.S. Army Corps of Engineers has imported rock from a distance of 170 miles. However, Reclamation has gained confidence in soil-cement slope protection. Because hauling costs have risen, soil-cement would probably be used instead of options for the long haul rock riprap.
This preference is based on the assumption that cost factors would favor the soil-cement option.

Dumped rock riprap of marginal quality has been used by increasing the layer thickness requirements and stockpiling a supply of riprap during construction for future maintenance. Soil-cement deserves serious consideration for upstream slope protection where the use of riprap is too expensive. Design of soil-cement slope protection is presented in Design Standard No. 13, Chapter 17, "Soil-Cement Slope Protection." Roller-compacted concrete (RCC) can also be used for slope protection and is considered equally adequate to soil-cement. RCC placement techniques are similar to those used to place soil-cement, although the technology is somewhat different.

If economical, other types of upstream slope protection such as precast concrete blocks, asphaltic concrete, steel plates, and concrete paving can be considered. It is possible that slope protection and water barriers can be combined in the case of upstream membrane-type dams. An upstream membrane consisting of concrete, asphaltic concrete, or steel may allow steeper slopes and additional embankment economy. In most cases, rock riprap and soil-cement will be the most suitable and economical solution for zoned or homogeneous embankments.

2.2.5.3.4 Soil-Cement

In recent years, soil-cement as a facing material for earth dams has been found to be economical where suitable riprap is not available near the site. No unusual design features need to be incorporated into the embankment. Normal embankment construction procedures are used, with perhaps special care being taken to ensure a minimum of embankment consolidation and foundation settlement after construction. Figure 2.2.5.3.4-1 shows soil-cement slope protection used on Choke Canyon Dam in Texas.

Figure 2.2.5.3.4-1. Soil-cement slope protection at Choke Canyon Dam, Texas.

The soil-cement is generally placed and compacted in stair-step horizontal layers, as shown on figure 2.2.5.3.4-2 at Starvation Dam in Utah. This promotes maximum construction efficiency and operational effectiveness. Using typical embankment slopes of 2:1 H:V to 4:1 H:V, a horizontal layer width of 8 feet will provide minimum protective thicknesses of approximately 2 to 3.5 feet, respectively, measured normal to the slope. However, soil-cement can also be placed parallel to the upstream slope; this is usually referred to as the “plating” method. The plating method provides for placement of soil-cement in one or more layers parallel to the slope face. Case histories indicate that plating soil-cement has been placed on 3: 1 H:V slopes or flatter. It is unlikely that the compaction equipment can travel steeper slopes. Figure 2.2.5.3.4-3 shows an example of plating soil-cement used at Brock Reservoir in California.

Figure 2.2.5.3.4-2. Placement of soil-cement slope protection, Starvation Dam, Utah.

Soil-cement slope protection is discussed in detail in Design Standard No. 13, Chapter 17, “Soil-Cement Slope Protection.” As previously noted, RCC is considered equally adequate in quality to soil-cement for slope protection.

Figure 2.2.5.3.4-3. Placement of soil-cement on the interior embankment slopes of Warren H. Brock Reservoir, California, using the plating method of placement.

2.2.5.4 Downstream Slope Protection

If the downstream zone of an embankment consists of rock or cobble fill, no special surface treatment of the slope is necessary. Downstream slopes of homogeneous dams or dams with outer sand and gravel zones should be protected against erosion caused by wind and surface runoff using a layer of rock, cobbles, or sod. Because of concerns with burrowing animals and the difficulty of obtaining adequate slope protection using vegetative cover at many damsites, especially in arid regions, slope protection using cobbles or rock is preferred and should be used where the cost is not prohibitive. Figure 2.2.5.4-1 shows the downstream cobble slope protection at Jordanelle Dam. Layers 24 inches in normal thickness are easier to place; however, a 12-inch-thick layer usually affords sufficient slope protection. Often, this type of material can be obtained by separating oversized materials from borrow areas or aggregate processing.

If grasses or other vegetation are planted, those suitable for a given locality should be selected, and a layer of topsoil is usually required. The advice of an agronomist should usually be obtained to ensure success.

Vegetation that will conceal seeps, animal burrows, etc., should not be used. Exit surfaces to internal drainage layers should not be covered by vegetation. Any vegetative covers should be maintained in a condition that will not conceal deleterious conditions. Slopes should be flat enough to allow access for maintenance equipment. Usually, fertilizer and uniform sprinkling of the seeded areas are necessary to promote the germination and growth of grasses.

Figure 2.2.5.4-2 shows the native grasses which have protected the downstream slope of Reclamation's Scoggins Dam from erosion for more than 35 years.

Figure 2.2.5.4-1. Cobble slope protection on downstream slope of Jordanelle Dam, Utah.

Figure 2.2.5.4-2. Vegetated slope protection on downstream slope of Scoggins Dam, Oregon.

2.2.5.5 Surface Drainage

The desirability of providing facilities to handle surface drainage on the abutments and valley floor is often overlooked in the design of earthfill dams. As a result, unsightly gullying may occur at the contact of the embankment with earth abutments, especially if the abutments are steep. Vegetation near the abutment contact is either removed purposely or unavoidably during construction operations; this exacerbates the erosion problem.

This condition is most likely to develop along the contact of the downstream slope with the abutments or along the upstream slope-abutment contact on dams with large normal freeboard. Gullying can usually be controlled by constructing a gutter along the contact. The gutter may be formed of cobbles or rock used in the downstream surfacing. If the downstream slope is seeded, a concrete, asphalt, or dry-rock paved gutter should be provided. The likelihood of gullying of the slopes of the dam or gentle slopes of the valley floor by runoff from the downstream slope of the dam should also be considered; contour ditches or open drains may be needed to control erosion. Figure 2.2.5.5-1 shows a photograph of a typical contour ditch, and figure 2.2.5.5-2 shows a typical section of a contour ditch and an open drain.

Figure 2.2.5.5-1. Contour ditch at Belle Fourche Dam, South Dakota.

Attention should also be given to the construction of outfall drains or channels to convey the toe drain or toe trench seepage away from the downstream toe of the embankment, so that an unsightly boggy area will not be created that adds to the difficulty of monitoring seepage. The need for surface drainage facilities and the most appropriate type for a particular site can usually best be determined by field examination prior to or during construction.

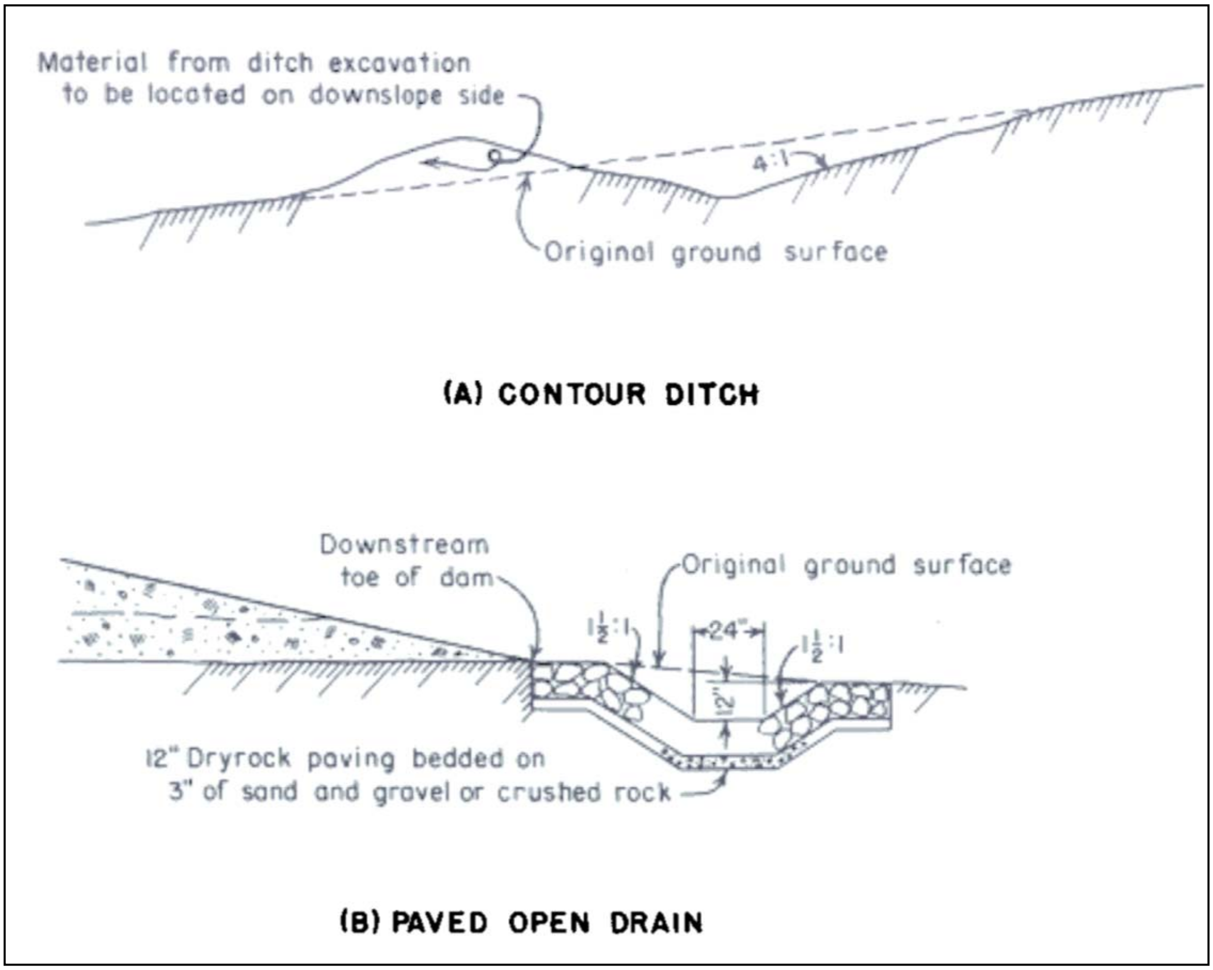


Figure 2.2.5.5-2. Typical sections of a contour ditch and an open drain.

2.2.5.6 Flared Slopes at Abutments

The upstream and downstream slopes of the embankment may be flared at the abutments to provide flatter slopes for abutment stability, to control seepage by providing a longer contact of the impervious zone of the dam with the abutment, or to provide an impervious cover over a pervious abutment. If the abutment is pervious, and if a positive cutoff cannot be attained economically, it may be possible to obtain the effect of an upstream blanket by flaring the embankment or orienting the centerline crest of the dam upstream of the ridge line to provide some impervious covering. A filter zone between the impervious zone and pervious foundation may be required in such cases. The design of the transition from normal to flared slopes is governed largely by the topography of the site, the length of contact desired, and the desirability of making a gradual transition without abrupt changes for ease of construction and for appearance.

2.2.5.7 Typical Maximum Sections

Reclamation earthfill dams feature a number of different outer slopes and internal zoning. Maximum sections of Reclamation dams are shown in "Maximum Sections and Earthwork Control" [24].

2.3 Rockfill Dams

2.3.1 Origin and Usage

Rockfill dams originated during the California Gold Rush, over 100 years ago [25]. From the late 1800s to the middle 1930s, many rockfill dams were constructed; the design and construction of a number of these dams are described by Galloway [26].

Interest in constructing rockfill dams diminished after the 1930s because of the increased costs of obtaining and placing large amounts of rockfill material, although a number of large dams were constructed in the 1950s [27]. Rockfill dam construction has increased since 1960 and is attributed to the utilization of more remote sites, more economical quarrying and placing operations, the use of excavated material in random zones, better design details, increased general knowledge concerning rockfills, and the advent of pumped storage projects in mountainous terrain. Recent advances in design and construction of rockfill dams are discussed by Cooke [28]. The excellent performance of an increasing number of rockfill dams has further stimulated their use.

Rockfill dams can prove to be economically favorable when any of the following conditions exist:

- Large quantities of rock are readily available or will be excavated in connection with the project, such as from a spillway or tunnel.
- Earthfill materials or concrete aggregates are difficult to obtain or require extensive processing to be used.
- Short construction seasons prevail.
- Excessively wet climatic conditions limit the placement of large quantities of earthfill material.
- The dam is expected to be raised at a later date.

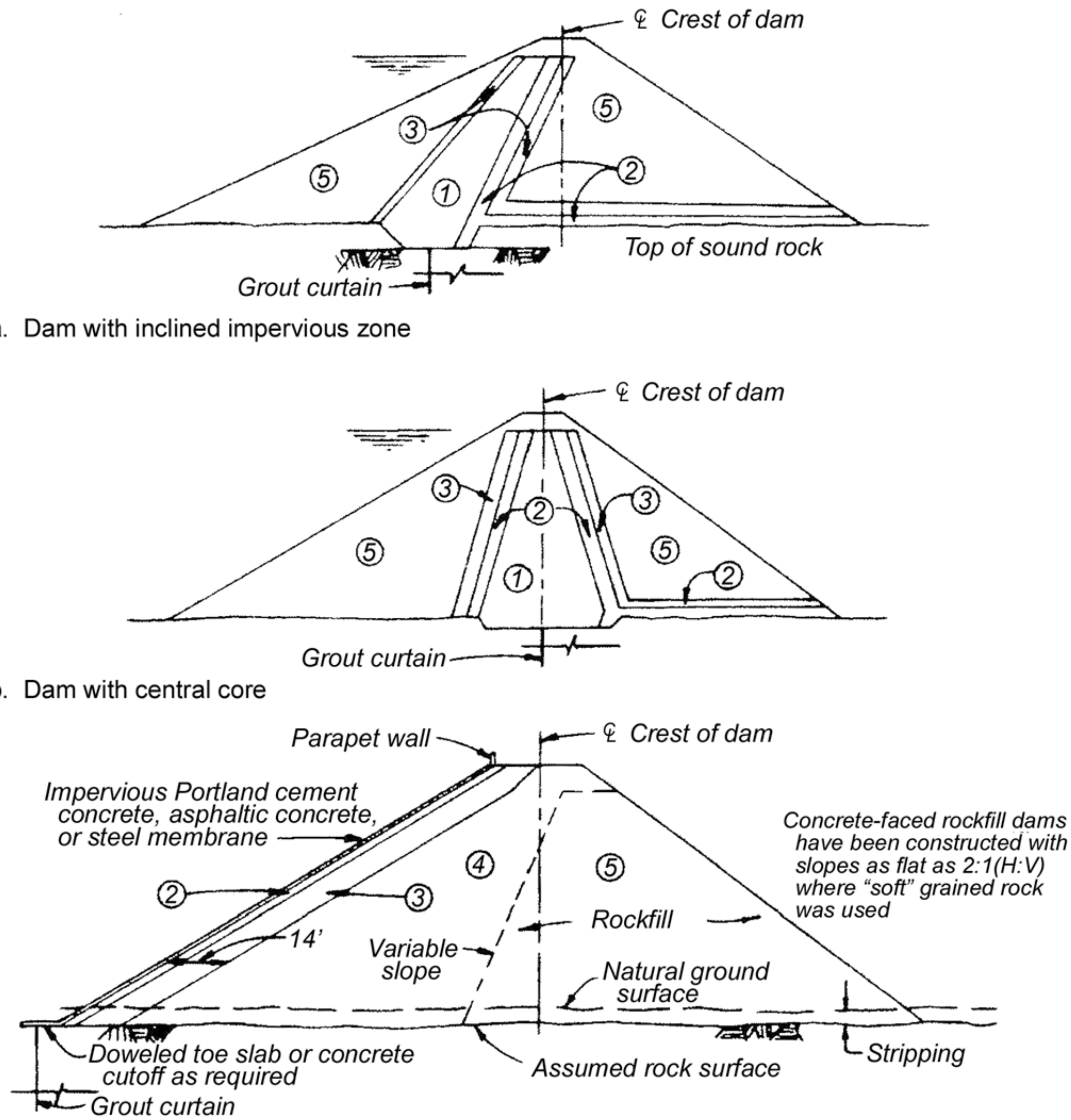
Other factors that may make use of a rockfill dam advantageous are the ability to place rockfill throughout the winter in cold regions, the possibility of grouting the foundation while simultaneously placing the embankment, and a high degree of seismic stability. In addition, uplift pressures and seepage through the rockfill material do not generally present significant design or operational problems (however, seepage losses could be an economic concern).

2.3.2 Definition and Types of Rockfill Dams

Rockfill dams have been defined as follows: "A dam that relies on rock, either dumped in high lifts or compacted in relatively thin layers, as a major structural element" [29, 30]. This standard has a further qualification that "rock" shall include angular fragments such as those produced by quarrying or occurring naturally as talus and subangular or rounded fragments such as coarse gravel, cobbles, and boulders [31]. It should be noted that dumping in high lifts has been essentially replaced by compacting in relatively thin layers. An impervious membrane is used as the water barrier and can be placed either within the embankment or on the upstream slope. Various materials have been used for this membrane including earth, reinforced concrete, steel, asphaltic concrete, geomembrane, and wood.

Rockfill dams may be classified into three groups, depending on the location of the membrane, as follows: (1) central core, (2) sloping core, and (3) upstream membrane or "decked." Figure 2.3.2-1 shows example cross sections of these different types of rockfill dams. However, both concrete and asphaltic concrete diaphragms are used both as internal and upstream membranes. Asphaltic concrete is used routinely in some European countries. Each membrane location has its advantages and disadvantages, which vary according to the type of membrane, materials available at the site, and foundation conditions. Central and sloping cores are referred to as "internal membranes," and these are generally constructed of impervious earth materials. Economic analyses should be performed to determine the type of material to use in constructing the membrane, either internal or upstream (external). If an internal membrane of impervious earth is to be used, there are no clear advantages to using a central vertical core versus an upstream sloping core. The choice will generally be based on economics and site-specific conditions. Refer to pages 35-37 of *Earth and Earth Rock Dams* [32] and page 31 of *Current Trends in Design and Construction of Embankment Dams* [33] when considering a vertical or sloping core.

ROCKFILL DAM SECTIONS



a. Dam with inclined impervious zone

b. Dam with central core

c. Dam with upstream membrane

Note: Sections a. and b. are modified from those presented in Fig. 1-2, EM 1110-2-2300, Dept. of the Army, Corps of Engineers, May 10, 1982. Section c. is modified from Fig. 202, Design of Small Dams, USBR, 1977.

1 - Impervious earth fill
2 - Filter zone
3 - Well graded, selected compacted rock used to provide drainage and bearing support for membrane, and/or transition
4 - Smaller sized rock from quarry and rock of lesser quality from foundation excavations, compacted to reduce membrane settlement, and/or provide transition
5 - Best quality, higher strength rock, compacted to provide section stability

Figure 2.3.2-1. Types of rockfill dams.

When comparing an internal membrane to an upstream membrane, the following advantages of each should be considered:

Internal Membrane

- Shorter grout curtain length because of straighter alignment.
- Protection from the effects of weathering and external damage.
- If the core is centrally located, any future remedial grouting can be accomplished from the crest; this is also true for cores that slope only slightly upstream.
- More easily adapted to less favorable foundation conditions, especially if the core is centrally located.
- Typically would not require specialized construction that may be needed for upstream membrane.

Upstream Membrane

- Readily available for inspection and repair if reservoir can be drawn down.
- Membrane can usually be completed during or after completion of the rockfill section.
- Foundation grouting is not on the critical path for embankment construction.
- Future remedial grouting can be accomplished if a gallery is included at the upstream toe.
- A larger portion of the embankment remains unsaturated, which is favorable for both static and dynamic stability.
- More mass of the embankment is available for stability against base sliding; see figure 2.3.2-2.
- Membrane also provides slope protection.
- More adaptable for construction in wet or cold climates because membrane and filters do not have to be placed simultaneously with the rockfill as they do for internal impervious core dams.
- With the downstream portion of the embankment essentially unsaturated and strong, failure of this type of dam is difficult to envision.
- Easier to raise this type of dam later.

If an external upstream membrane is used, it is recommended that it be constructed of concrete or asphalt. The reservoir should be capable of being drawn down to an elevation which will permit inspection and repair; video cameras or audio devices may be used for leak detection, and minor repairs may be made by divers.

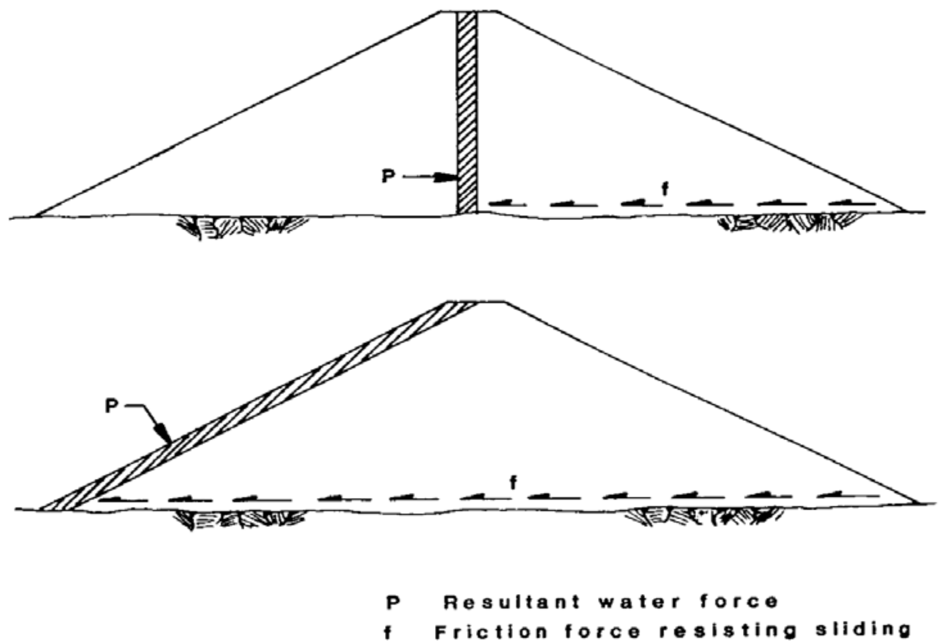


Figure 2.3.2-2. Effect of upstream membrane on embankment resistance to sliding.

If an earth-core rockfill dam is used, it requires the use of adequate filters both upstream and downstream of the core. The downstream filters should satisfy the requirements listed in Design Standard No. 13, Chapter 5, "Protective Filters," and Chapter 8, "Seepage." A critical function of the upstream filter zone is to serve as a "crack stopper," and though it can be designed to less stringent requirements, it should have 5 percent or less material passing the No. 200 sieve and contain only cohesionless fines. The upstream filter zone must also prevent material from being removed from the impervious core during reservoir drawdown. If adequate earth material for either the core or the filter material is not available at the site, and processing to obtain impervious or filter materials is required, the earth-core rockfill dam may be uneconomical due to processing costs. Construction costs of the earth-core rockfill dam will also increase significantly if several filter layers are required to ensure filter compatibility between the core and the rockfill shells.

2.3.3 Impervious Elements Other than Clay Cores

It is sometimes advantageous, or possibly necessary, to construct the impervious element of a rockfill or earthfill dam using materials other than soil. The primary reasons for using alternative materials are the lack of suitable impervious soil, climate, and cost savings. Advanced technology in both design and construction of alternative membranes is dramatically increasing their use. Extensive coverage of alternative materials was given in the Sixteenth Congress on Large Dams [34]. Basic concepts for design of alternative materials are presented in the following sections of this chapter, but engineers considering the use of such materials as alternatives to earth cores should also refer to the proceedings from the Sixteenth Congress on Large Dams [34].

2.3.4 Foundation Design for Rockfill Dams

2.3.4.1 Foundation Requirements and Treatment

Foundations which consist of hard and erosion resistant bedrock are the most desirable. The use of foundations consisting of river gravels or rock fragments is acceptable under some circumstances, but a positive cutoff to rock is usually necessary. The foundation should be selected and treated so as to result in minimum settlement of the rockfill embankment. Any materials in fractures or deep excavations which may eventually erode into the rockfill, either from the foundation or from the abutment, should be protected with filters or removed, if necessary, and backfilled with concrete or suitable backfill. If an earth core is to be used, the foundation should be treated in accordance with Design Standard No. 13, Chapter 3, “Foundation Surface Treatment.” The “central contact area” beneath filters and transitions should generally receive the same treatment as the impervious core foundation, unless those zones are needed to drain foundation seepage at that specific location.

The alignment of the dam should be selected so that either minimum embankment volume or minimum membrane exposure is attained, depending on which criterion is economically more important, or a combination of the two, as long as the design takes advantage of existing topography and geology, and as long as foundation conditions are adequately considered and treated.

Foundation treatment must be sufficient to satisfy the following criteria:

- Minimize leakage
- Prevent internal erosion
- Prevent settlement that will cause divergence between abutment and fill, or large discontinuities
- Sufficient friction development between the embankment and its abutments and foundation to ensure base sliding stability

In the past, designers of decked rockfill dams (dams with an impervious element on the upstream face) have felt that all soil deposits should be removed beneath the structure. The current philosophy would allow competent soil deposits to remain under the downstream one-third of the embankment, and where very competent deposits such as dense sands and gravels exist, more of the deposit has been left in place. The rationale is that with the seepage barrier further upstream, the piezometric surface is low, and the sliding resistance is greater under much of the embankment. However, the stability of the embankment, and the liquefaction potential of the foundation materials, should be evaluated thoroughly before a decision is made to leave soil or less competent rock formations in place. **It is also essential that the liquefaction potential of the soil deposit be carefully evaluated before adopting this approach.**

2.3.4.2 Membrane Cutoffs

For safe and efficient operation, it is critical to prevent seepage beneath the dam and to obtain a watertight seal between the membrane and the foundation. To reduce seepage beneath the dam, foundations are usually grouted. Determining whether grouting is required, and its extent, should be based on careful study of the site geology, a visual examination of the drill cores from the rock foundation, and drill-hole water tests. If no data are available, it should be assumed that grouting will be required; however, where reservoirs are completely drawn down each year, grouting requirements can be based on seepage observations over the first few years' operations.

Cutoff walls (essentially the “plinth” when a concrete deck is the membrane) are excavated to various depths into bedrock to prevent leakage in the upper part of the foundation, to facilitate grouting operations, to provide a watertight seal with the membrane, and to resist the downward thrust of the membrane. Figures 2.3.4.2-1, 2.3.4.2-2, and 2.3.4.2-3 illustrate typical cutoff wall details. Drainage galleries are sometimes used in conjunction with cutoffs or plinths to facilitate future grouting and to determine seepage locations and quantities.

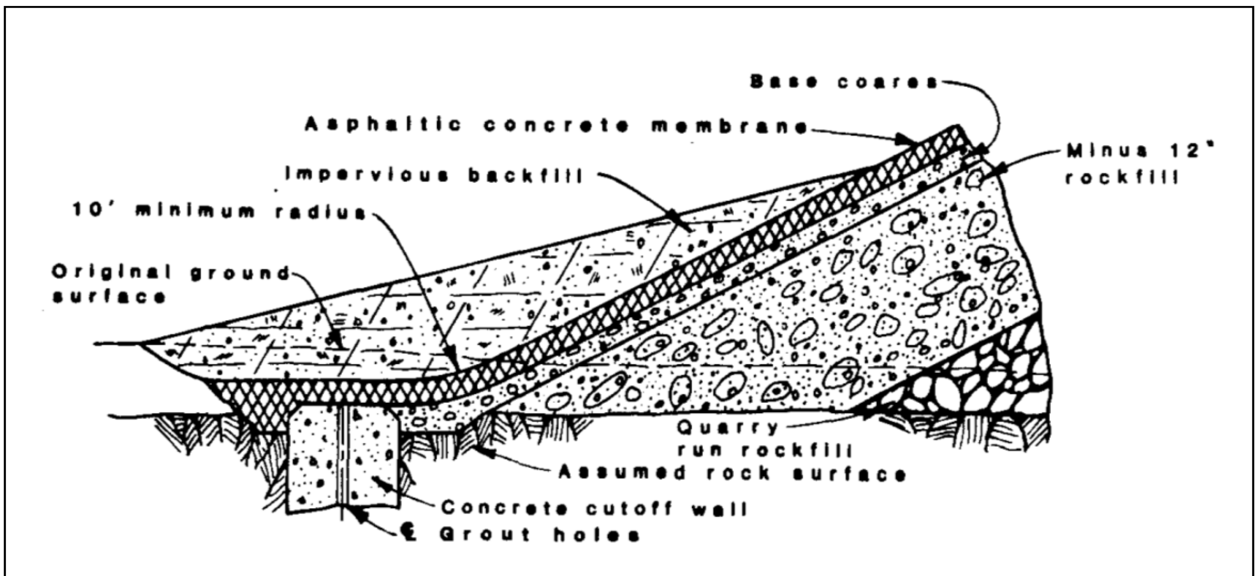


Figure 2.3.4.2-1. Detail of asphaltic concrete membrane at cutoff wall.

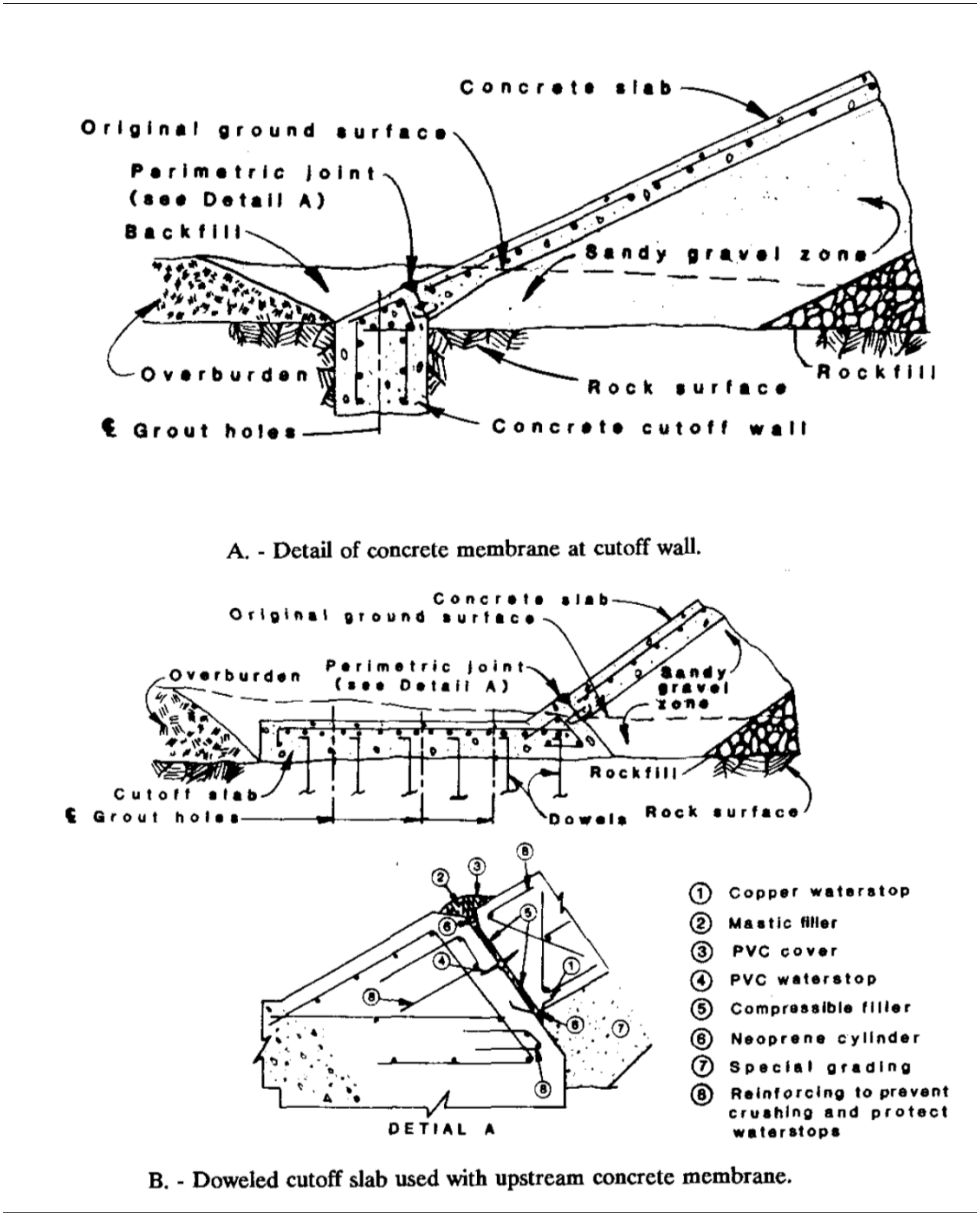


Figure 2.3.4.2-2. Details of (a) concrete cutoff wall, and (b) doweled cutoff slab for a concrete membrane.

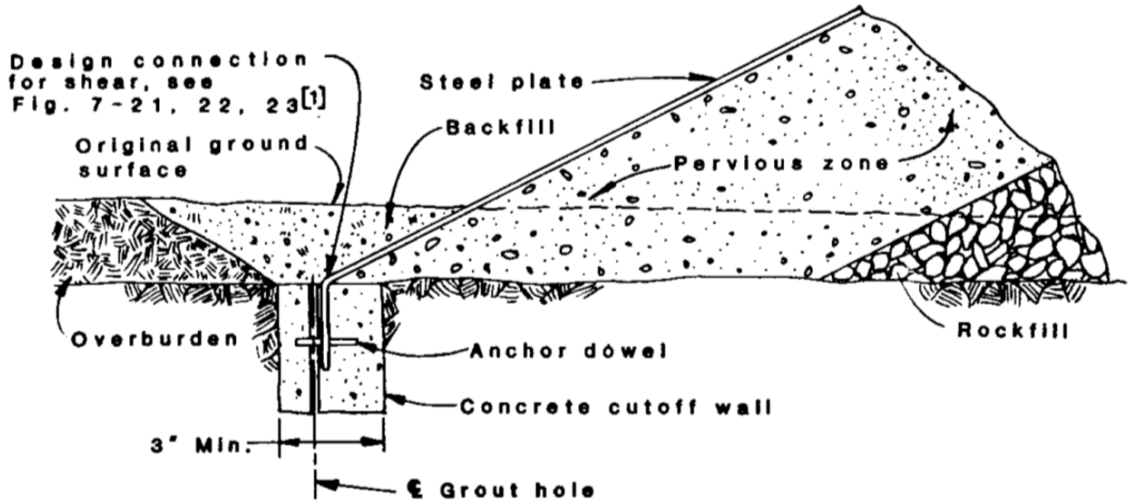


Figure 2.3.4.2-3. Detail of steel plate membrane at cutoff wall.

Doweled toe slabs (or plinths), as shown on figure 2.3.4.2-2 (B), have been used in conjunction with concrete facings to provide the foundation-membrane seal [28, 35]. Doweled toe slabs have the advantage of not requiring extensive excavations in rock, thereby preventing damage to the foundation and allowing grouting operations to begin earlier, speeding completion time, and reducing costs. Toe slabs can be used where the bedrock is strong and nonerodible, and where few underseepage problems are expected. Surface treatment of the rock beneath the toe slab should be similar to that required beneath the impervious zone of an earth-core dam. When uncertainty concerning the permeability of upper portions of the foundation contact exists, such as the presence of soft or fractured rock, a cutoff wall into bedrock, as discussed in the previous paragraph, can provide increased protection and allow examination of questionable material. Any blasting should be done carefully to minimize damage to rock outside the cutoff wall.

A minimum width and depth of 3 feet is recommended for cutoff walls in sound rock, and they should be deepened depending on the depth and intensity of weathering of the rock foundation. The width of the doweled toe slab is based on the depth of water, height of dam, foundation conditions, construction, or grouting requirements. The minimum width has generally been 10 feet. If poor bedrock conditions are present, a wider toe slab may be required. Multiple row grout curtains are easiest to construct beneath a doweled toe slab. In addition to their function of preventing leakage, both the cutoff wall and the doweled toe slab must be designed to provide adequate support for the thrust of the membrane and, in the case of steel membranes, any tension imparted to the cutoff due to embankment settlement. The possibility of leaving the lower edge of the steel membrane free until initial settlement of the embankment occurs should be considered.

The cutoff should extend along the entire upstream contact between the membrane and the foundation.

2.3.5 Embankment Design for Rockfill Dams

2.3.5.1 Selection of Rock Materials

A great variety of rock types have been used in constructing rockfill dams. The types of rock used range from hard, durable, granite, and quartzite to weaker materials such as greywacke, sandstone, and slaty shale. In the past, designers thought that only rockfill material of the highest quality should be used; however, with the advent of thinner lifts and more efficient compaction techniques, rock of less desirable characteristics has been used within the embankment sections. A well-graded mixture of rounded gravels and cobbles are the most durable type of rockfill because more angular rock under high stress levels tends to break at the contact points of the rock particles or along fractures, causing more settlement and more deformation. The use of rock from excavations for spillways, outlet works, tunnels, and other appurtenant structures has reduced the construction cost of rockfill dams without impairing the embankment's usefulness or stability. If small amounts of the less desirable rock types are available, they can be used in random zones within the embankment; the use of material in random zones is discussed in Subsection 2.2.4.3, “Utilization of Materials from Required Excavation.”

Rock material should preferably be hard, durable, resistant to weathering and wetting, and be able to resist excessive breakdown due to quarrying, loading, hauling, and placing operations. Figure 2.3.5.1-1 shows the metamorphic serpentine rockfill on the downstream face of New Melones Dam in California. The rock should also be free of unstable minerals that would weather mechanically or chemically and cause the rock to disintegrate. Igneous, metamorphic, and sedimentary rocks have all been used successfully in embankment sections, and only general advice can be given concerning rock types because each damsite will present unique considerations for using the nearby rock materials.

Laboratory tests which measure the abrasion resistance, freeze-thaw characteristics, and percent of water absorption can be used to evaluate rock sources and types for suitability of the rockfill material. Petrographic and x-ray diffraction analysis can be used to distinguish minerals known to weather easily. Unconfined or triaxial compression tests can be used to evaluate the strength properties of the rock. One of the best methods to determine a rock's resistance to weathering is simply to examine its *in situ* condition; however, this does not always indicate how the material will perform within the fill after saturation. Materials available at the site should be examined by constructing test embankments, especially in cases where the material properties are questionable. Test fills can determine the following items:

Figure 2.3.5.1-1. Rockfill on the downstream slope of New Melones Dam, California.

- Whether or not marginal materials can be used
- How selected embankment material will perform during compaction operations
- Suitable type of compaction equipment for each material
- Required number of passes of equipment used for each material
- Appropriate lift thickness for each material
- The necessity for changing the embankment section to accommodate new materials or different material properties

As an example, Crisp [36] reports that significant design changes in Carters Dam were proposed because of results obtained by placing test embankment sections of quartzite, phyllite, and argillite.

Test fills should be constructed using equipment and methods that the contractor is likely to use.

The effect of quarry blasting methods on the gradation of the rock, percentage of oversize rock, and need for processing should also be examined, as well as the required extent of quarrying.

Also of importance to the design engineer when evaluating the rock fill material is the degree to which small-scale triaxial compression tests will provide strength parameters applicable to the actual rockfill material. Large-scale triaxial compression tests are very expensive. Fortunately, researchers have made data available on this subject. Marachi et al. [37] examined this problem by testing 36-, 12-, and 2.8-inch diameter specimens in drained triaxial compression tests using parallel grain-size curves and similar grain shapes (modeling) to examine the effects of grain size on the strength and deformation characteristics of rockfill material. Also investigated was the effect of particle crushing.

Three types of material were tested as follows:

- Pyramid Dam: Argillite, a fine-grained, sedimentary rock, quarry-blasted, angular, with relatively weak particles (G_s = 2.67)
- Crushed basalt: Quarry blasted and crushed to the correct size, angular, and quite sound (G_s = 2.87)
- Oroville Dam: Amphibolite, a metavolcanic rock, rounded to subrounded particles with some subangular fine sand particles, river-dredged material, hard (G_s = 2.86 to 2.94)

Figure 2.3.5.1-2 shows the gradation curves for the actual rockfill material and the modeled material.

Although the report was primarily concerned with the use of rockfill material in high dams, the following general conclusions are applicable to rockfill dams of all sizes:

- Rockfill materials can be successfully modeled so that the strength and deformation characteristics of the actual material can be obtained from small-scale tests.
- At any given confining pressure, as the particle size of the specimen increases, the angle of internal friction decreases a small but significant amount.
- Rockfill materials composed of well-graded and well-rounded particles are superior to uniformly graded angular rockfill materials, especially for high dams.
- For any given particle size, as the confining pressure of the sample increases, the angle of internal friction decreases.

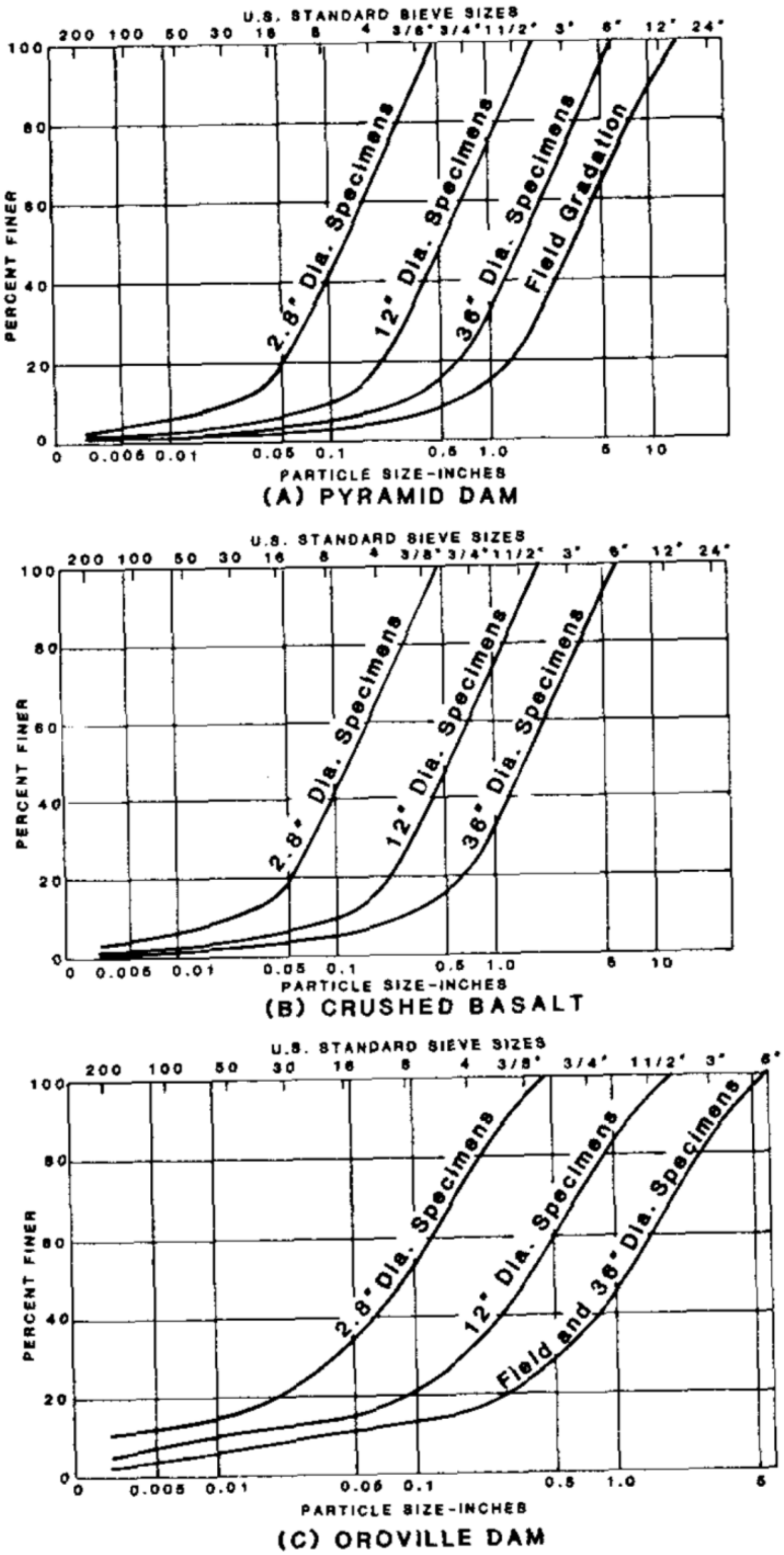


Figure 2.3.5.1-2. Grain size distribution for modeled rockfill materials.

Figure 2.3.5.1-3 illustrates the variation of the angle of internal friction with both particle size and confining pressure. The general reduction in friction angle with increasing confining pressure and particle size at constant confining pressure shown in this figure should be of interest to designers.

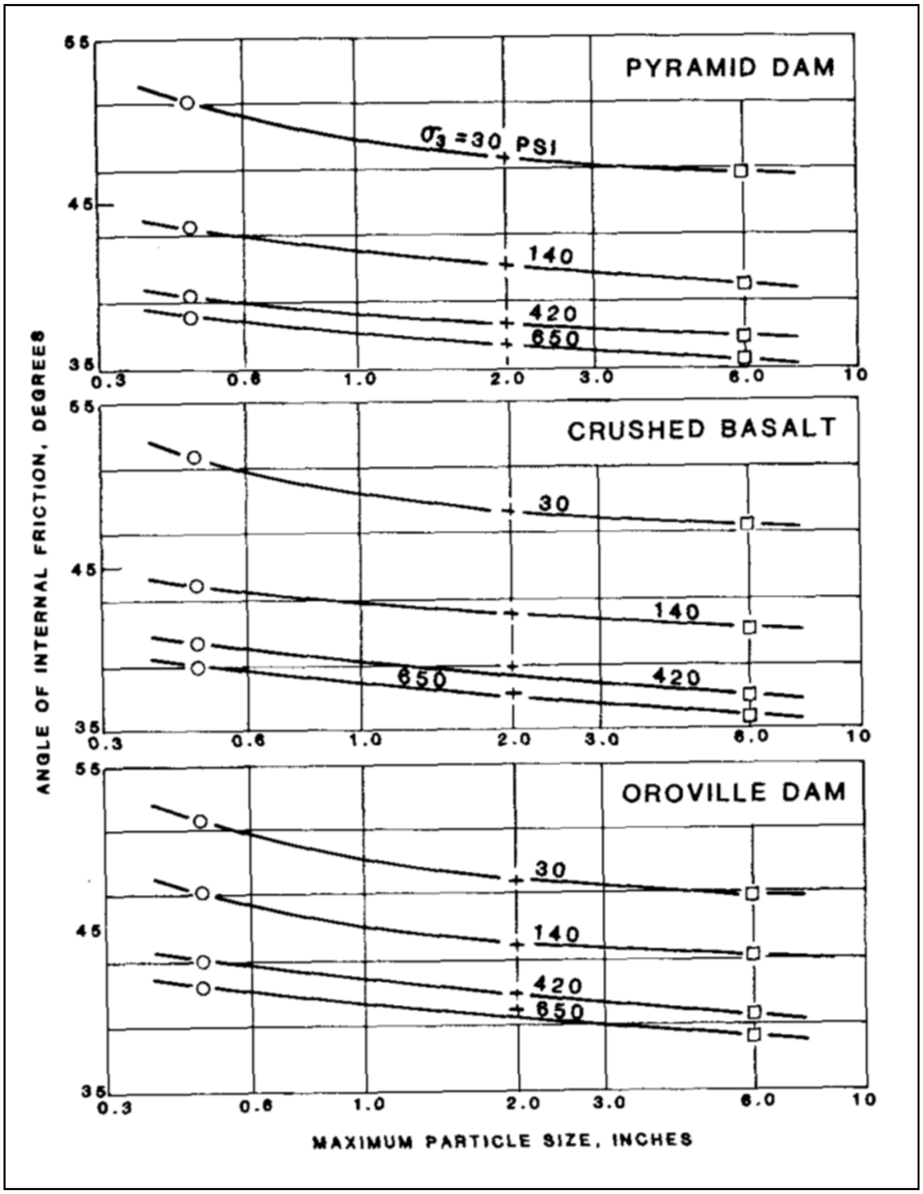


Figure 2.3.5.1-3. Effect of maximum particle size on the angle of internal friction. Adapted from Marachi et al. [37]

Refer also to the report by T.M. Leps [38] for the details of testing and for further conclusions regarding the strength and deformation properties of rockfill materials and the crushing characteristics of rock subjected to high confining pressures. These results are summarized on figure 2.3.5.1-4. Finally, laboratory testing of rockfill was performed by the University of California – Berkeley and documented in a 1972 report [39].

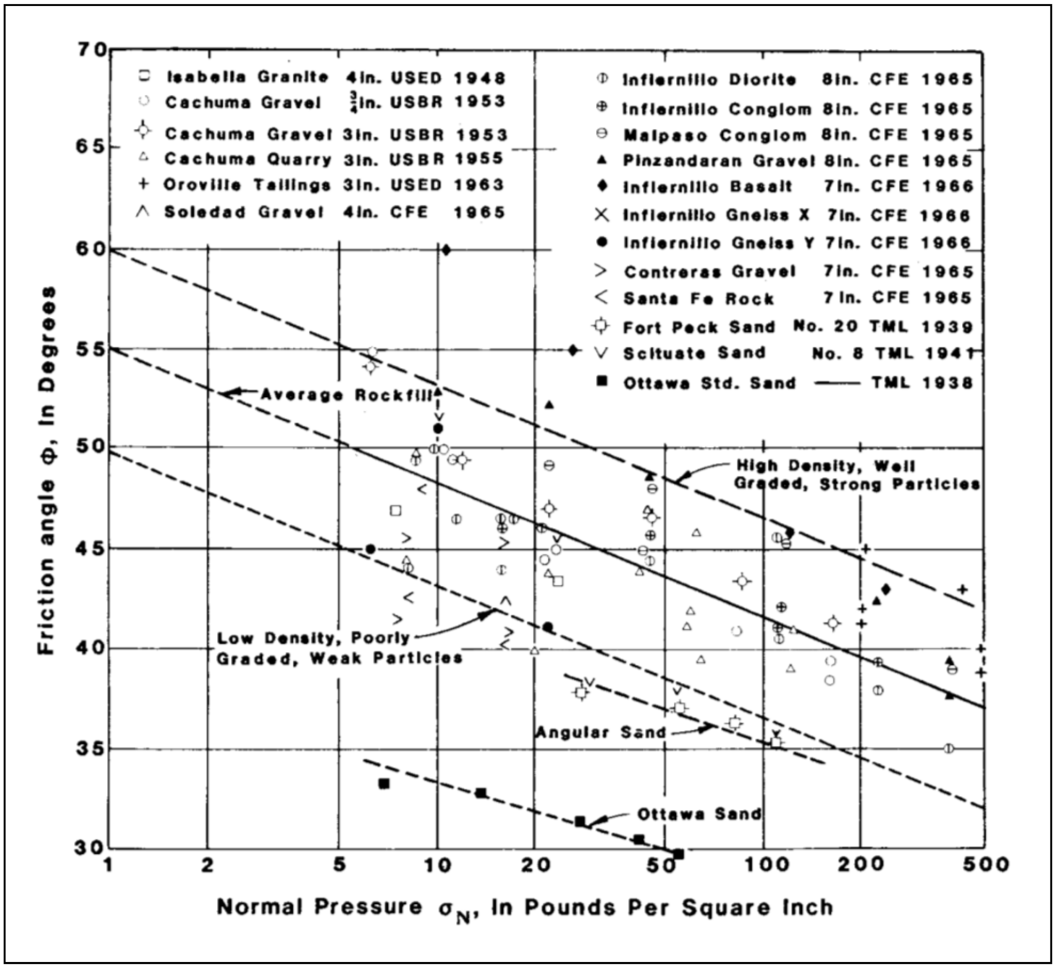


Figure 2.3.5.1-4. Shearing resistance of rockfill from large triaxial tests [38].

There are other publications and literature that the engineer can use as guidelines for selecting shear strength for rockfill. An appropriate strength can generally be selected from available information. If the engineer has doubts about the appropriateness of strength selected from graphs presented herein, or from other sources, a testing program may be appropriate.

2.3.5.2 Embankment Sections for Rockfill Dams

Embankment slopes used for rockfill dams have evolved from very steep slopes, usually 0.5 to 0.75:1 H:V, which were used on early rockfill dams, to the flatter slopes of 1.3:1 to 2.0:1 H:V used in current practice. Slopes of 1.3:1 to 1.4:1 H:V roughly correspond to the angle of repose of loose dumped rockfill and prevent raveling of the embankment slopes. If gravel or weaker rock is used for the rockfill zone, flatter slopes may be required to prevent raveling of the embankment slope. The strength of the foundation material should also be considered when designing the embankment slope. Foundation strengths may dictate flatter slopes. Earlier rockfill dams used upstream membranes exclusively and were constructed with steep upstream and downstream slopes to minimize the volume of rockfill. Because these slopes were considerably steeper than the natural slope of dumped rock, they were stabilized by thick zones of crane-placed, dry-rubble masonry, which provided the bedding for the upstream facing. The rockfill portions of these dams were constructed by dumping and sluicing the rockfill in thick lifts which ranged from 30 to 165 feet. Later designs eliminated the rubble masonry on the downstream slope by flattening it to the angle of repose of the rock, but the very steep upstream slope was retained. Because most of the upstream zones were constructed by crane placement of large rocks, the cost of the dams continually increased. Designers found that it was more economical to use slopes approximating the angle of repose of the dumped rock material and eliminate crane placement. Gradually, because of excessive deformation (especially in higher dams), dumped rockfill was replaced by compacted rockfill.

The upstream and downstream slopes of the dam depend on the type of impervious membrane and its location. Rockfill dams having central or sloping earthfill cores usually have slopes of about 1.5:1 to 2:1 H:V upstream and downstream, often depending on the location of the core. The upstream slope is generally flatter, particularly for upstream sloping cores. Rockfill dams having a thin membrane that is placed on the upstream face usually have upstream slopes of from 1.3 to 1.7:1 H:V and downstream slopes that approximate the angle of repose of the rock.

Most asphaltic concrete-faced dams have been constructed with upstream slopes of 1.6 to 1.7:1 H:V to facilitate construction of the membrane, and most steel and concrete-faced rockfill dams have used slopes of 1.3 to 1.4:1 H:V. Available literature indicates that these slopes have performed satisfactorily. Advances in technology may allow use of steeper slopes for asphaltic concrete dams [34].

The upstream and downstream slopes for central or sloping earth-core rockfill dams depend on the size and soil properties of the earth core, the width of filter zones required, type of foundation material, drawdown requirements, construction sequence, etc., with each site presenting its own unique design considerations. Figure 2.3.2-1 shows typical embankment sections for earth-core rockfill dams. Actual embankment sections are shown in Maximum Sections and Earthwork Control Statistics [24].

Figure 2.3.2-1.c. also shows typical zoning for a decked rockfill dam. Note that similar zones may be used in rockfill dams with internal cores, with the zones 2 and 3 potentially designed as filters and with zones 4 and 5 comprising the bulk of the rockfill shells. The zoning number designations for the decked rockfill dam shown on figure 2.3.2-1.c correspond to the zoning descriptions discussed below, rather than to the zoning descriptions shown on figure 2.3.2-1.c. The interior section of the decked rockfill dam can be divided into four zones and can be described as follows:

> **Zone 2:** Well-graded sand and gravel used to provide a base course for the membrane, a leveling course, and a good working surface.
>
> **Zone 3:** Well-graded, smaller sized rock and gravel used to provide support for the upstream membrane and a transition from zone 2 to 4. Zone 3 will retard extreme water loss if the membrane cracks or joints open to the extent that sealants and waterstops become ineffective.
>
> **Zone 4:** Smaller sized rock than that used in zone 5, such as high quality rock from required excavation, used to minimize cost. The compressibility of zone 4 must be low enough so that detrimental movement in the membrane does not occur; ideally, the horizontal permeability of this zone is high to allow drainage of any leakage.
>
> **Zone 5:** The larger downstream zone of the dam consisting of high quality, larger sized, compacted rock. This zone provides high downstream stability to the section; ideally, the horizontal permeability of this zone is high to allow drainage of any leakage.

Placement conditions for these four zones are discussed in Subsection 2.3.5.4, “Placement of Rockfill Materials,” below.

Gradation requirements are difficult to specify because they depend on the type of rock available and the quarrying methods used. As with many aspects of dam design, only general rules apply. However, as a general statement, filter criteria specified in Design Standard No. 13, Chapter 5, “Protective Filters,” must be satisfied. High quality rock is desirable for decked rockfill dams. The downstream zone 5 of the embankment should use the largest rock available. Large slabby rocks (length-to-width ratio greater than 3:1) should not be placed in the fill because they tend to bridge, causing large voids, which may result in excessive settlement if the rocks break. If possible, rock in zone 5 should be well graded in size from a maximum size of approximately 1 cubic yard. The fines content should be low to ensure satisfactory permeability. Optimally, zone 4 should be well graded from a maximum size of approximately 10 cubic feet and have high permeability after compaction. Zone 3 should be well graded from approximately 3 inches to 5 to 15 percent passing the No. 100 sieve. If zone 2, as described later, is not necessary, the gradation of zone 3 will depend on the type of facing used and its method of construction. If zone 2 is not used, zone 3 material should provide a smooth uniform bearing surface for the facing and a gradation that retards large water loss if the facing cracks.

Zone 2 may not always be necessary, depending on the need for a leveling course and the gradation of zone 3 and zone 3's ability to withstand erosion caused by rainfall prior to placement of the deck or raveling during placement of the deck. In any event, the zone immediately beneath the face slab should provide a good working surface for equipment and workmen during placement of the facing element, retard extreme water loss, and resist erosion during surface runoff [40]. A light application of asphalt emulsion is often applied to the surface to improve the constructability and resistance to erosion. Zone 2 should be well graded from a maximum size of 3 inches with 2 to 12 percent passing the No. 200 sieve [40].

In general, material in zones 4 and 5 should grade from fine rock upstream to coarse rock downstream, with the largest and strongest material placed in the lower downstream portions of zone 5. Selection of the rock for each zone should be made at the quarry. Rockfill embankment slopes are usually selected on the basis of experience and a design requirement that they not become saturated. Therefore, it is paramount that the fill be free draining and of adequate quality.

For central earth-core rockfill dams, the larger and stronger rock should again be placed in the outer rockfill zones and grade from fine rock next to the filter to coarse rock near the outer slope.

The centerline of the dam crest may be either curved (convex upstream) or straight. A curved crest will act to compress the dam as filling occurs, whereas a straight crest has the benefit of easy construction layout and less total dam cost. For small dams, given good foundation and abutment conditions, it is recommended that a straight crest be used. It is also recommended that for upstream-membrane rockfill dams, the layout should allow a minimum area of membrane face to be exposed. This expedites face construction and reduces face and cutoff cost, as well as repair costs, if they become necessary.

Random zones constructed of rock that has questionable strength or permeability characteristics may also be used within the rockfill embankment to increase economy if the stability of the section is not compromised and adequate drainage to prevent saturated zones is provided. Test embankments can be used to determine whether or not materials will be adequate.

Crest width will be determined by its use after construction and by the type of membrane used. It should, however, be of sufficient width to accommodate construction of the upstream membrane; a minimum width of at least 20 feet is recommended. Crest camber should be determined by the amount of foundation and embankment settlement anticipated. A value of 1 percent of the embankment height is sometimes used; guidelines are given in Design Standard No. 13, Chapter 9, “Static Deformation Analysis.” A straight line equation may be used to define the crest camber as illustrated in figure 2.2.5.1.4-1. Additional considerations concerning crest details were given previously in Subsection 2.2.5.1, “Crest Details,” as discussed for earthfill dams. Freeboard requirements will depend on maximum wind velocity, fetch, reservoir operating conditions, slope roughness, spillway capacity, etc. Freeboard determinations should be in accordance with Design Standard No. 13, Chapter 6, “Freeboard.”

If coping or parapet walls are used to prevent overtopping by wave runup and splashover, freeboard requirements may be reduced from those normally required for a riprapped embankment. If coping walls are not used, the freeboard should be adequate to prevent wave runup from flowing over the crest. Economy in the use of rockfill can be achieved by using a high quality coping wall and letting it support the upper portion of the embankment. The wall should be started at a level of the embankment where the width needed for concrete face slip-forming equipment and material supply is available. The quantity of rockfill placed on the upstream face of the dam can be reduced by using higher coping walls. The coping wall must be designed to be stable against the load of the fill placed against it. Good results have been obtained with coping walls, and their use is recommended [35].

2.3.5.3 Stability

Experience and judgment are important in determining the stability of a rockfill dam. If the rockfill material in an embankment does not consist of high-quality rock, or if there are weak foundation zones, stability analyses are necessary. However, if fill materials are strong and competent, and the foundation is competent, an infinite slope stability analysis may be sufficient. Both static and seismic stability should be considered and documented, including rationale used in cases where stability analyses were limited. Consideration of the variation in rockfill strength with confining pressure may be important for stability analyses of high dams [37, 38]. Design Standard No. 13, Chapter 4, “Static Stability Analysis,” and Chapter 13, “Seismic Design and Analysis,” should be used in performing stability analyses.

2.3.5.4 Placement of Rockfill Materials

Limiting settlement to acceptable limits is critical in constructing rockfill dams because excessive settlement may rupture the upstream membrane or cause joint separation, with subsequent water loss for rockfill dams with upstream membranes. For central core rockfill dams, excessive settlement could cause differential settlement between the core and the rockfill shells, leading to potential drag, low-stress zones, and cracking. Early rockfill dams were constructed by placing the rock in high lifts; it was assumed that the height of drop imparted compaction energy to the fill, decreased the embankment's void space, and, thus, reduced settlement. Experience has shown that dumped rockfill dams often settle and deflect downstream significantly during initial filling. Consequently, many of these high lift embankments have developed leakage problems, and experience has indicated that rock material placed in thin lifts (1 to 4 feet thick) and compacted by vibratory rollers provides a more stable mass in which settlement is minimal. For decked rockfill dams, the embankment should preferably be completed before construction of the upstream membrane begins because this reduces the probability of serious membrane cracking by allowing initial settlement to take place.

Settlement of rockfill material has been correlated with the application of water, and Sowers, et al [41] have shown that some dumped rockfill material placed dry and subsequently wetted may settle appreciably. In many cases, water is not necessary to obtain adequate compaction and its use for that purpose would be wasteful. However, in some rockfill, it is necessary, and sufficient water should be added to the rockfill to facilitate compaction and settlement during compaction. Test fills and compression tests should be used to determine the need for water to facilitate compaction. This is a critical cost consideration in arid and semi-arid regions. Good compaction of rockfill materials not only minimizes total settlement but also minimizes differential settlements between zones that have significantly different consolidation characteristics.

2.3.5.5 Compaction

Figure 2.3.2-1 shows typical sections of rockfill dams. The zone 5 material should be sound, durable rock of high quality, typically dumped in 2- to 4-foot lifts (depending on rock size), and compacted by a vibratory roller. Zone 4 material may consist of smaller rock than that used in zone 5 (such as spillway excavation or tunnel spoil) and should be dumped in 2- to 3-foot lifts and compacted by a vibratory roller. However, the compression modulus of zone 4 must be sufficiently high to prevent settlement from cracking in membranes of decked rockfills. Zone 3 material provides the bearing surface for the upstream membrane or transition for the impervious core and may be either a processed or selected material from quarry or borrow pit excavations. Zone 3 material should be compacted to 12-inch lifts by vibratory rollers; if water is necessary for compaction, the material should be thoroughly wetted prior to compaction. Zone 2 is usually used as a working surface and leveling course for decked dams, as well as a filter for core zones. If zone 2 is used as a sloping layer in a decked dam, it should be compacted in accordance with zone 3 requirements by rolling on the slope. Suggested gradations for zones 2, 3, 4, and 5 were discussed previously in Subsection 2.3.5.2, "Embankment Sections for Rockfill Dams."

The size of the vibratory roller used for each rockfill zone depends on the properties of the rock used in that zone and should preferably be established by constructing test embankments. Vibratory rollers from 3 to 20 tons have been the most widely used for rockfill compaction. The larger roller would be used for thicker lifts and larger rock sizes, and the smaller roller might be used for compacting the face of a decked rockfill, thinner lifts in transition zones, or where access is difficult. The face of the zone 3 material should be compacted by drawing a smooth drum vibratory roller up and down the slope. Generally, the vibrator is turned off for the first two passes to prevent displacement. If zone 2 is used beneath the deck, it should also be compacted by drawing a smooth-drum vibratory roller up and down the slope. As with zone 3, the vibrator would be turned off for the first two passes.

For central or sloping earth-core rockfill dams, the upstream and downstream rockfills should be compacted in (typically) 2- to 4-foot lifts by vibratory compactors to provide the most stable section possible. If necessary, as previously discussed, the fill should be thoroughly wetted to facilitate compaction.

For all types of rockfill, the lift thickness and compaction effort ultimately selected should depend on considerations such as maximum rock size, specified rollers, tolerable settlements, seismic concerns, and similar factors.

Compaction of rockfill zones is discussed more thoroughly in Design Standard No. 13, Chapter 10, “Embankment Construction.”

2.3.6 Membrane Design for Rockfill Dams

2.3.6.1 Impervious Core

Figure 2.3.2-1 shows typical earth-core rockfill sections using central and sloping impervious earth cores. Sloping cores of impervious earth materials are sometimes advantageous from placement sequence and/or availability of materials considerations. Internal membranes of concrete, asphalt, and steel have also been used and are sometimes advantageous. In the past, earth cores have been favored over internal membranes because the relative thinness and brittleness of membranes causes them to be more likely to rupture. However, recent developments have resulted in the availability of “plastic” concrete and asphaltic mixtures that are less brittle. The inability to inspect and repair internal membranes is also a disadvantage. The rockfill zones of the internal core dam were discussed previously in Subsections 2.3.5.2, “Embankment Sections for Rockfill Dams,” and 2.3.5.5, “Compaction.” The upstream rockfill material should be of sufficient size and quality to satisfy riprap requirements as discussed in Design Standard No. 13, Chapter 7, “Riprap Slope Protection.”

Earth-core rockfill dams are economical where impervious fill is locally available and climatic conditions favor placement. The impervious material used in the core should be similar to the material used for earthfill dams, as discussed in Section 2.2.2.1.3, “Zoned Embankments.” The material should be placed near optimum moisture content and compacted in thin lifts as discussed in Design Standard No. 13, Chapter 10, “Embankment Construction.” The plasticity index of the material should be sufficient to allow the core to deform without cracking. Measures should be used to prevent the core and adjacent material from settling at different rates and amounts that could result in low stresses and cracking in the core. The use of transition zones with less compaction than the shells is one method of accomplishing this. The hydraulic gradient across the core contact with the foundation should also be considered. A minimum of one-fourth the hydraulic head is often referenced in the literature, but this depends on type and availability of impervious materials, adequacy of filters and drains, quality of foundation rock, surface treatment of foundation rock, etc.

Filter zones should be adequate to prevent internal erosion of impervious material during steady-state or rapid drawdown conditions. The filter criteria in Design Standard No. 13, Chapter 5, “Protective Filters,” should be used for designing downstream filters and drains. Consideration can be given to relaxing the filter criteria for upstream transition zones. Multiple filters may be required if gradation differences between the core and rockfill materials are large. Figure 2.3.6.1-1 shows the placement of sand filter and gravel drain material at Spring Canyon Dam in Colorado.

Figure 2.3.6.1-1. Placement of sand filter and gravel drain material at Spring Canyon Dam, Colorado.

The foundation and abutments upon which the core is placed should be carefully treated as prescribed in Design Standard No. 13, Chapter 3, “Foundation Surface Treatment,” to prevent internal erosion. Freeboard requirements are the same as that required for earthfill dams and are discussed in Design Standard No. 13, Chapter 6, “Freeboard.”

2.3.6.2 Reinforced Concrete

A larger number of rockfill dams have been faced with reinforced concrete than with any other type of impervious membrane. In most cases, these facings have performed well, for well-compacted rockfill embankments, with acceptable limits of leakage and minor repairs. Slab thickness and reinforcing requirements have usually been determined by experience or precedent, with the goal of satisfying the following criteria:

- Low permeability
- High resistance to weathering action
- Sufficient strength to bridge subsided areas of the face
- Sufficient slab articulation to tolerate embankment settlements

The significance of the last two items has diminished somewhat with the advent of compacted rockfill dams.

Compaction of rockfill has considerably reduced embankment settlement, and the use of a well-compacted facing layer, which acts as a continuous, firm bedding surface for the concrete face, has reduced the bridging requirements of the face. The concrete slab should be reinforced and have a minimum thickness of 12 inches. An equation that has become a rule of thumb for calculating slab thickness at a point on the face is $\mathbf{t} = 1 + 0.003\ \mathbf{h}$ in feet, where $\mathbf{h}$ is the height of dam in feet above the point on the face [28, 42]. The concrete should be dense, durable, weather resistant, and of low permeability. The concrete should have a compressive strength of 3,000 to 3,500 pounds per square inch.

Higher strength concrete is generally not desirable because it tends to have a higher cement content with a greater tendency to shrink and crack. In areas subject to extreme weather conditions, consideration should be given to using measures to prevent freeze-thaw damage, such as air entrainment and pozzolan in the concrete. If foundation settlement may occur, or if other factors such as earthquake conditions exist, the slab should be designed to withstand these forces.

Steel reinforcement is provided to control cracking due to temperature and shrinkage. In general, the face slab is in compression. The amount of steel reinforcing should meet the generally accepted requirement of 0.4 to 0.5 percent of the concrete area in each direction. The single layer of reinforcing should be placed in the center of the slab.

The face slab is placed in strips by continuous slip-forming methods. Hand-placed starter slabs are usually necessary at the bottom perimeter of the slab. Figures 5 and 6 of reference [28] show the construction sequence of face slab placing. Horizontal joints are not necessary, except for constructability, where a simple joint with reinforcing going through the joint should be used. Vertical joints are cold joints having polyvinyl chloride (PVC) or rubber waterstops that are used to ensure impermeability along the joints. Where horizontal, cold construction joints are used, careful placement is required to prevent honeycomb concrete. Care must also be exercised at vertical joints to ensure correct installation of waterstops and placement of dense concrete. Good supervision and careful placement of concrete are required at all joints so that watertight performance is attained. The chemistry of reservoir water should also be considered when selecting waterstops. More aggressive water chemistry may require alternative materials or more redundancy. Multiple waterstops (copper, rubber, and PVC) are currently used at the contact of the face slab and plinth or toe block (parametric joint).

The type of cutoff between the concrete facing and the foundation will depend on the quality of rock encountered. For sound rock, the doweled cutoff shown on figure 2.3.4.2-2(B) has demonstrated its adequacy and economy [35], whereas in closely jointed, weathered rock, or rock of questionable quality, the cutoff wall should be used. Waterstops should be used between the cutoff and facing. Rigid cutoffs are not recommended because they restrict the allowable settlement of the face.

Because concrete facings provide little resistance to wave runup, increased freeboard is required to prevent oversplash. Coping or parapet walls can be used to reduce the height of embankment required for freeboard purposes; these walls should be constructed as integral continuations of the concrete face and be reinforced accordingly. Walls should only be used in the freeboard range. Appropriate analysis should be made to design the walls for imposed loading conditions. The designed top of the rockfill should be above maximum water surface. Camber should be provided to ensure that the design crest does not settle below maximum water surface. Camber should be built in under the footing of parapet walls. Design Standard No. 13, Chapter 6, “Freeboard,” should be consulted to determine freeboard requirements.

Concrete placement is generally by the same slip-forming process used in road or canal construction. Figure 2.3.6.2-1 shows the placement of concrete using slip forms on the upstream slope of New Exchequer Dam in California. Preferably, placement of the concrete membrane should not begin until the entire embankment has been placed, which allows for maximum construction settlement and reduces the possibility of cracking and excess leakage. If concurrent slab placement is necessary, design allowances should be made for settlement during construction.

Publications referencing the design and construction of concrete-faced rockfill dams are the proceedings of an American Society of Civil Engineers (ASCE) symposium, “Concrete Faced Rockfill Dams - Design, Construction, and Performance” [43], held in Detroit in October 1985, and two dozen discussions and closures to papers from this symposium that are published in the *Journal of Geotechnical Engineering*, ASCE, October 1987 [42]. The *Journal of Geotechnical Engineering* also contains two additional papers: one gives a general assessment of concrete-faced rockfill dams, and the other provides a review of design details. These two references present a comprehensive practical reference work on concrete-faced rockfill dams. Engineers involved in the design of rockfill dams should refer to these publications.

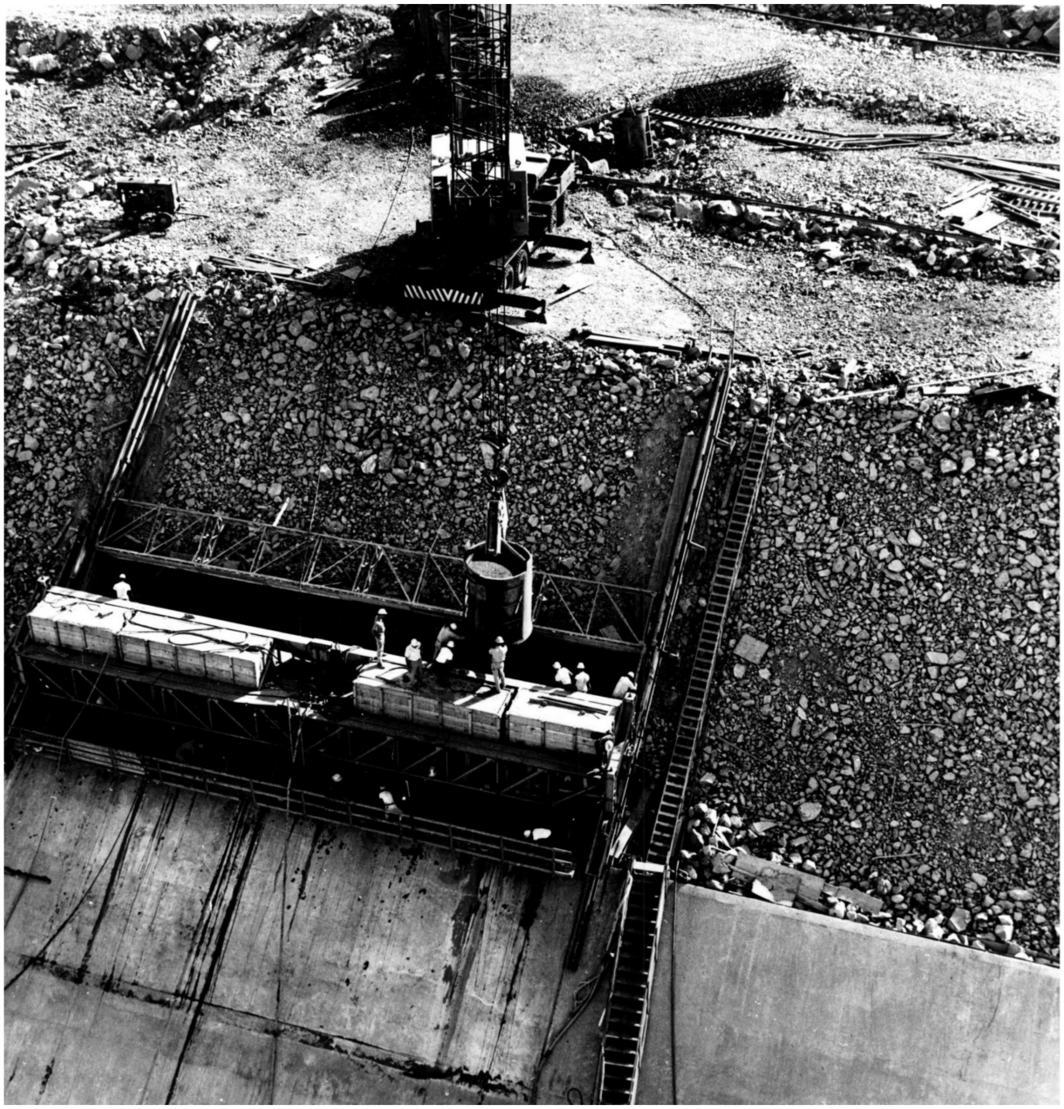


Figure 2.3.6.2-1. Placing concrete using slip forms on the upstream slope of New Exchequer Dam, California. (Tudor Engineering Company)

2.3.6.3 Asphaltic Concrete

The second most widely used facing for rockfill dams is hydraulic asphaltic concrete. Hydraulic asphaltic concrete provides more flexibility and, thus, can tolerate larger settlements than reinforced concrete facings. It offers an economical alternative to concrete and has proven to be dependable when correctly constructed. Hydraulic asphaltic concrete has a higher percentage of minus No. 200 material, a higher percentage of asphalt, and a lower voids content than roadway paving asphalt. The upstream slope for asphalt-faced rockfill dams is recommended to be 1.6:1 or flatter H:V, although asphalt facing has been placed on upstream dam faces up to 1.1H:1V using modern construction equipment and methods [55]. The zone 3 material should provide a well-graded, free-draining rock layer to eliminate uplift pressures in case of rapid drawdown It should also provide sufficient resistance to limit water velocities and prevent internal erosion if a crack forms in the membrane. The gradation of this material should be smaller than zone 4 material. A base course should be provided beneath the asphalt to provide a leveling course, working surface, and smooth base surface for asphalt placement. The base course should be well compacted by a vibratory roller. Figure 2.3.6.3-1 shows the completed rockfill section at Upper Blue River Dam prior to membrane placement.

Figure 2.3.6.3-1. Completed rockfill embankment at Upper Blue River Dam, Colorado, during membrane placement. (Photo courtesy of Department of Public Utilities, Colorado Springs, Colorado.)

A penetration coat should be applied to the base course surface prior to membrane placement to bind and stabilize it. The weight of the paving machine may still gouge the base course, and hand placement of asphaltic concrete in the gouged surfaces may be required.

Single asphaltic concrete membrane thicknesses typically range between 2 and 4 inches, depending on the hydraulic head, and are thicker in some cases. Two layer membranes have also been used and can feature a second asphalt mix that has a much higher hydraulic conductivity and is installed beneath the primary liner. This second layer acts as a draining layer in conjunction with granular material beneath the lining system. Hydraulic asphalt is applied by a paving machine in one to two approximately equal lifts, depending on the total thickness [44]. Paving machines, as well as milling and other equipment, are held on the upstream face using winches. Figure 2.3.6.3-2 shows placement of an asphaltic concrete layer at Montgomery Dam in Colorado.

Figure 2.3.6.3-2. Placing asphaltic concrete on the upstream slope of Montgomery Dam, Colorado.

A seal coat is desirable on the finished surface of the membrane. The seal coat protects the facing from ultraviolet light and oxidation degradation. Each layer is placed in strips approximately 10 to 12 feet wide, depending on the equipment used, and constructed at right angles to the axis of the dam, except in areas such as adjacent to the plinth, where hand placement and special compaction methods are often required. Paving is placed on the upslope pass only, and the machine is reloaded at the top or bottom of the paving run, depending on the site staging. If the paving machine hopper capacity is insufficient and an adequate volume of asphaltic material is not available for each strip, reloading must be performed on the upstream face. In recent years, procedures have been developed to pave long faces horizontally. Horizontal operations require winches that are capable of traveling across the dam crest while, at the same time, supporting paving and other equipment. Staging of equipment on the dam crest for milling or paving operations can lead to crowded conditions and difficult logistics. If possible, lowering an embankment crest will provide extra staging room for construction activities. Rolling operations should closely follow the placing operation to ensure that the compaction occurs while the hydraulic asphalt is at a suitable temperature. Smooth wheel rollers, either of the vibratory or standard tandem type, can be used for layer compaction.

Layers should be compacted to a minimum of 97 percent of Marshall density.[2] Quality control can be achieved by taking cored samples of placed asphalt at random locations from the face and performing asphalt content, density, stability, and permeability tests, as well as any tests required by the specifications. A density that ensures an air-voids content of 3 percent or less, as discussed later, should be obtained.

Obtaining tight joints between adjacent strips of the facing is important to the imperviousness of the membrane. Transverse asphalt joints should be kept to a minimum, if possible, and should be constructed as hot joints. Cold joints, either between parallel strips or oriented transversely on a single strip, should be treated as follows:

1. Apply a tack coat of mastic. This is often the same type used in the mix design.

2. Place the asphaltic concrete tightly against the cold joint to ensure that no voids remain at the contact.

3. Reheat the joint with an infrared heater, avoiding overheating, burning, or open flames.

4. Compact the joint by rolling, immediately following reheating.

When one layer is placed on top of another, the parallel joints in the strips of the top layer should be offset 3 to 4 feet from the joints of the bottom strip to prevent paving lane joints from stacking vertically.

The foundation cutoff used with asphalt facings must promote easy placement of asphalt at the contact with the concrete plinth. Figure 2.3.4.2-1 shows a trench-type cutoff wall. More massive plinths, anchored atop a prepared bedrock foundation, and of ample size to accommodate foundation grouting activities, have been successfully constructed. Figure 2.3.6.3-3 shows the cutoff used at Montgomery Dam in Colorado. The 12-inch-diameter drain was used to reduce uplift pressure beneath the lining, particularly during drawdown. The cutoff used at Upper Blue River Dam is visible at the left edge of figure 2.3.6.3-1.

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[2] The Marshall density test helps in determining stability and flow characteristic of the asphaltic concrete mix, as well as density, air voids of the mix, or voids in the mineral aggregate. Reference for this particular test is American Society for Testing of Materials (ASTM) D6927-06.

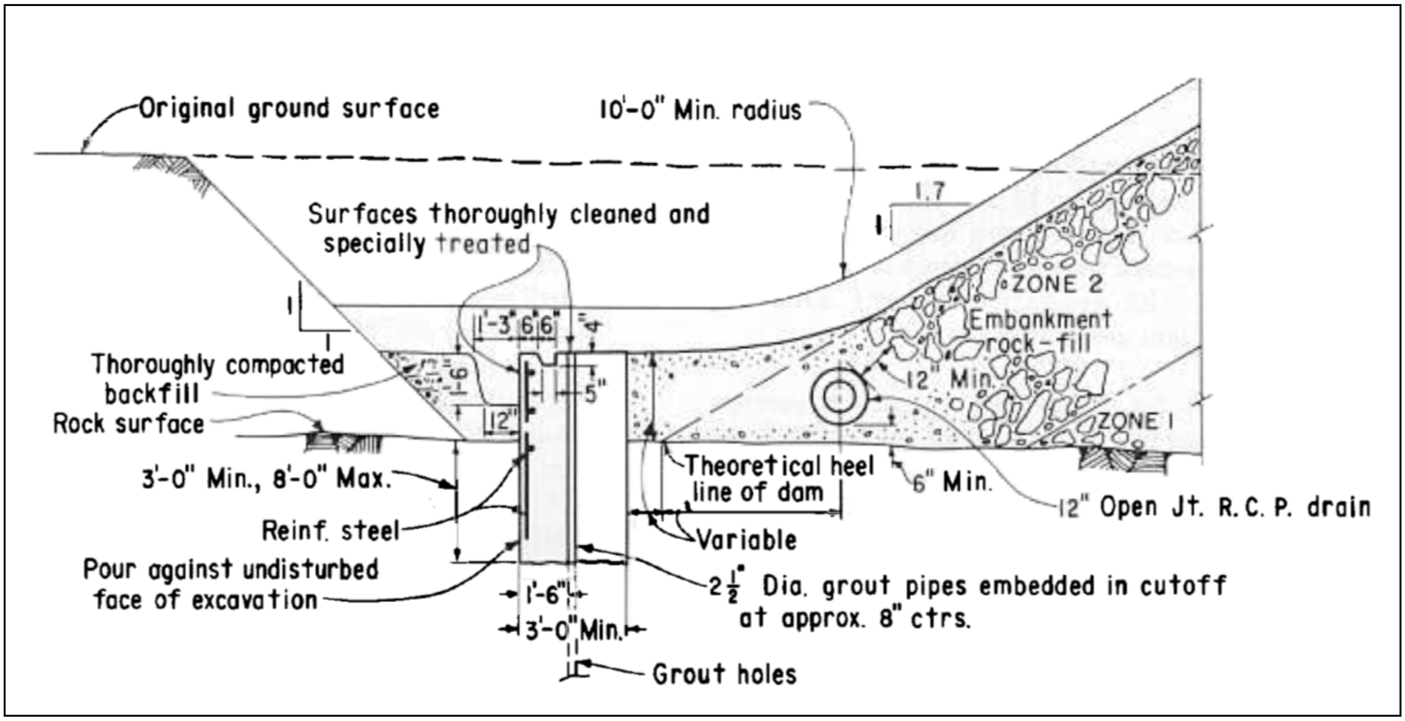

Figure 2.3.6.3-3. Foundation cutoff used at Montgomery Dam, Colorado.

The upstream asphaltic membrane should be constructed so that it is:

- Stable
- Durable
- Flexible
- Impervious
- Does not crack
- Does not creep
- Resists weathering

Material within an economical distance of the dam should be used in the asphaltic concrete mix, if possible; however, given the very specific aggregate needs required to create hydraulic asphalt, it is uncommon to find a construction site capable of producing appropriate materials without significant processing. When the project site does have appropriate aggregate materials, onsite processing will commonly be required. It is also common to import materials, particularly the fine mineral fillers. A number of different materials and gradations ranging from silty sand to graded gravel [34, 45, 46, 47] have been used to construct upstream facings. Clay materials should not be permitted in mixes because the clay tends to ball during the drying process and to crush when compacted, thereby leaving dry material exposed to the reservoir water. Rock fines (also known as mineral fillers) are the preferable fines material.

Specifications for materials used to manufacture the asphaltic concrete are subject to change; up-to-date literature should be consulted. Asphalt facing projects are somewhat uncommon in the United States but are a more common construction practice in Europe.

A very low air-voids content resulting from proper mix design and compaction is required to obtain durable facings; however, a low air-voids ratio cannot be obtained simply by adding additional asphaltic cement. Fine materials such as mineral filler are needed to occupy void space within the asphalt mix. Fine mineral fillers must be thoroughly dried prior to addition to the mix to allow the fine material to be well distributed. Air-void ratios of 1 percent are commonly obtained in laboratory testing, and the maximum air-void ratio allowed during construction of an asphaltic facing should be 3 percent [48].

Several differing types of hydraulic asphalt have been used successfully at different projects around the world. Varying site conditions, including weather, locally available aggregates and asphaltic oils, and available equipment all contribute to what the final hydraulic asphalt mix design will be. The mix design process should be undertaken early in the project to ensure that the best hydraulic asphalt mixture is selected for the particular project. Use of laboratory testing and field placement tests are valuable in determining the most suitable mix. Selecting the appropriate asphaltic cement is important. Testing often includes penetration testing, viscosity testing, and ductility testing, among others. Additionally, the adherence of the asphaltic cement to the aggregate should be determined. If aggregates are not being well covered by the asphaltic cement, increased permeability may result.

Parapet walls should be used with asphaltic concrete facings to retard wave runup and oversplash, rather than increasing the height of the dam. Galvanized corrugated metal has been used for parapet walls for a number of small dams [46, 49], but over time, many of the welded connections in these types of parapet walls have failed due to thermal expansion and contraction of the metal plates. Precast concrete barriers, such as the commonly available Jersey Barrier, have been used successfully where minimal run-up is anticipated. Cast-in-place parapet walls, designed for specific site conditions, would provide the most robust protection. Figure 2.3.6.3-4 shows the parapet wall used at Upper Blue River Dam. When parapet walls are used to protect against wave runup and oversplash, the freeboard heights of the embankment may be reduced from those heights required for earthfill dams; however, the embankment crest must be above the maximum water surface (the parapet wall should not be used for impounding water). Wall heights can be determined by precedent or design experience, and they should adhere to any regulatory requirements associated with the project.

For further information on asphaltic facings, the reader should consult the references at the end of this chapter. The following list of references can provide additional useful information regarding asphaltic concrete facing design and construction: [34, 48, 50, 51, 52, 53, 54, 55, 56, and 57].

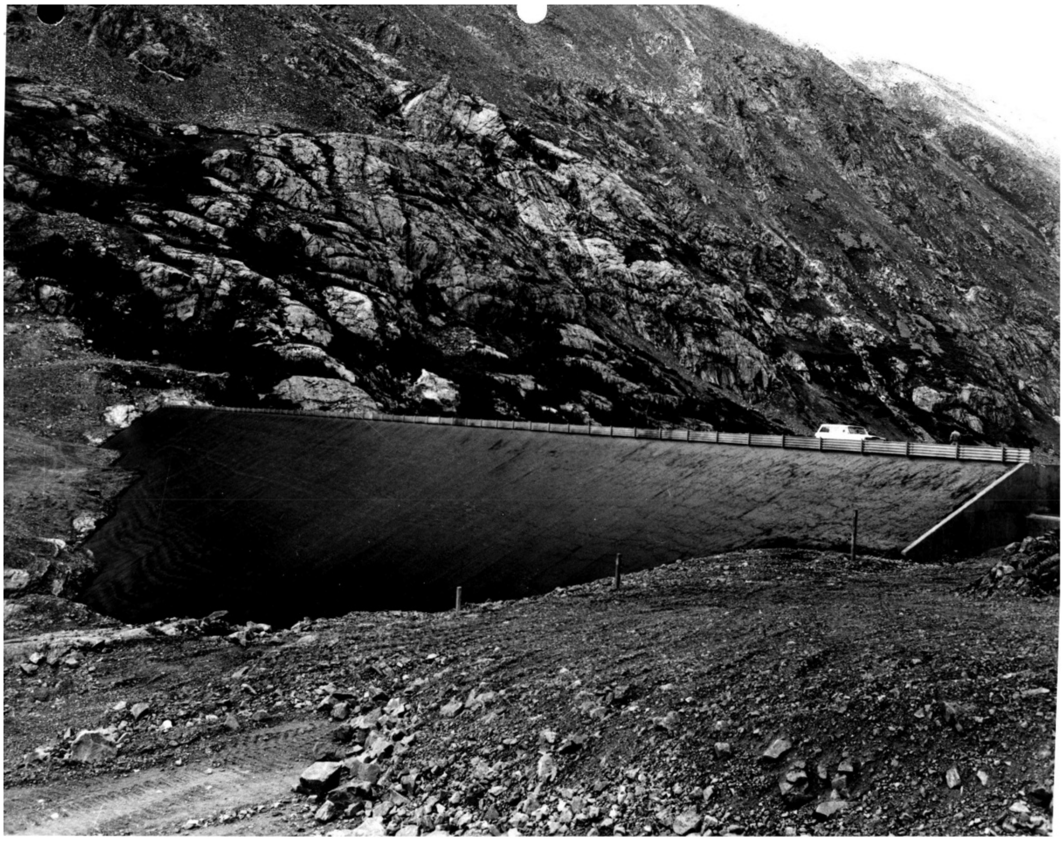

Figure 2.3.6.3-4. Completed asphaltic concrete facing at Upper Blue River Dam, Colorado. (Photo courtesy of Department of Public Utilities, Colorado Springs, Colorado.)

2.3.6.4 Steel Facings

Steel facings have been used on relatively few dams throughout the world. They are very adaptable for use in extremely cold climates. Few design criteria other than precedent are applicable, and the available literature should be consulted for a complete review of the practices used [32, 58, 59, and 60].

Figure 2.3.6.4-1 [61] is a photograph taken in 2007 of the upstream face of Reclamation's El Vado Dam in New Mexico; the steel plate continues to serve as an upstream diaphragm, although there are current concerns about its performance. After approximately 75 years of operation, the steel plate facing is manifesting signs of distress and deterioration. Cracking, as well as some separation at joints and pitting, is prevalent throughout the facing. In some cases, the pitting/corrosion has opened holes that penetrate the ¼-inch steel plating. The presence of an active abutment landslide has played a role in the distress noted at

El Vado Dam, but it would also appear that some of the facing deterioration is simply a function of a finite life span. There are no current plans to replace the facing.

Figure 2.3.6.4-1. 2007 photograph of upstream slope of El Vado Dam, New Mexico.

Steel-faced dams can be rapidly constructed and should be capable of tolerating greater embankment movements than either concrete or asphalt. The most prominent disadvantage to steel facings is the probability of corrosion, which reduces their economic life. However, this can be combated effectively by using cathodic protection on both faces of the steel plate. Experience with the few steel-faced dams now in existence strongly indicates that a corrosion failure of the steel plate is remote and that, for all practical purposes, the facing can be assumed permanent if proper maintenance is provided.

Steel-faced dams have generally been constructed with upstream slopes varying from 1.3:1 to 1.7:1 H:V. For rockfill dams, the upstream and downstream slopes need not be flatter than the natural slope of the material, which generally varies from 1.3:1 to 1.4:1 H:V. The steeper slopes lead to reduced costs but face construction difficulties will be slightly increased.

The portion of the embankment on which the steel plate bears should generally be constructed of well-graded, pervious gravel to provide a uniform bearing surface for the steel facing. Anchor rods should extend from the facing plates into the embankment to prevent uplift or loosening of the face due to embankment settlement or wave action. *Design of Small Dams* [62], chapter VII, paragraphs 7-12, discusses and shows details of steel plates, anchors, joints, foundation cutoffs, parapets, etc., for steel-decked dams.

2.3.6.5 Timber Planking

Timber planking has been used as a temporary type of membrane, but it is not recommended for general use, even though it is often the most inexpensive type of membrane to construct. The principal objections to this type of construction are the danger of loss by fire at low water and the relatively short life of timber construction when alternately exposed to wetting and drying.

2.3.6.6 Geomembranes

Although some still may question their long-term durability, geomembranes are gaining widespread use as impervious elements for dams (in earthfill embankments, as well as rockfill embankments). In general, geomembranes should be protected from exposure and be reasonably accessible for future repair. In addition, defensive design measures should be included to protect against any unexpected leakage. Geomembranes are discussed in Design Standard No. 13, Chapter 20, “Geomembranes.”

2.4 Evaluating and Modifying Existing Embankment Dams

This chapter has been focused on the recommended practice for designing new embankment dams, whether they are comprised of earthfill or rockfill. However, modifications to existing dams will frequently involve many of the same considerations involved in the design of a new dam. When raising an existing dam, or designing other modifications such as a downstream chimney filter/drain and buttress, it is important to follow the same practices as outlined herein for “new” components or portions of the reconstructed embankment.

It is important to note that most modifications to Reclamation structures are constructed to remediate dam safety issues. Therefore, it is important that the modification to an existing dam be based on mitigating the risks associated with specific potential failure modes. In these cases, compliance with standards such as the amount of recommended freeboard, recommended safety factors, and similar criteria may not be consistent with a risk-informed approach to lowering the risk at a given dam. It is important to recognize the importance of both standards and risk concepts. With any modification design at an existing embankment dam, the designer must carefully consider the benefits and drawbacks of these sometimes competing requirements. Ultimately, the strength of the case outlined in support of a particular criteria or concept will be a major factor in the design decision.

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