United States Society on Dams

Materials for Embankment Dams

January 2011

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Prepared by the USSD Committee on Materials for Embankment Dams

U.S. Society on Dams

Vision

To be the nation's leading organization of professionals dedicated to advancing the role of dams for the benefit of society.

Mission — USSD is dedicated to:

- Advancing the knowledge of dam engineering, construction, planning, operation, performance, rehabilitation, decommissioning, maintenance, security and safety;
- Fostering dam technology for socially, environmentally and financially sustainable water resources systems;
- Providing public awareness of the role of dams in the management of the nation's water resources;
- Enhancing practices to meet current and future challenges on dams; and
- Representing the United States as an active member of the International Commission on Large Dams (ICOLD).

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FOREWORD

The USSD Committee on Materials for Embankment Dams has prepared this *White Paper on Materials for Embankment Dams*. The paper provides an outline of important points that need to be recognized and understood when selecting material for use in embankment dams. It covers soil materials; rockfill materials; granular filters and drains; asphalt concrete as a water barrier; concrete facing rockfill dams; geosynthetics; reinforced fill; upstream slope protection; material for watertight cutoffs; and construction issues. For each of these topics, design and construction considerations are presented.

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CHAPTER 1 — INTRODUCTION

This white paper provides a brief summary of the use of materials in embankment dams. It is not intended to serve as an exhaustive treatise on the characteristics of the various materials that comprise these types of dams. It is, rather, an outline of important points that need to be recognized and understood when selecting materials for use in the embankment dam.

This introduction provides a framework within which materials for use in the dam are selected and evaluated. The basic requirements and functions of embankment dams are described, along with underlying concepts for their design, construction and successful operation.

1.1. HISTORICAL PERSPECTIVE

Embankment dams have served man at least 5,000 years. The remains of ancient structures and civilizations provide clues to the efforts of mankind in the engineering and construction of dams. Jansen (1980) traces the history of dams from the period BC to the 20th century. Of the earth dams built BC, Jansen comments:

> *"Turning to the most available materials, the ancient dam builders made liberal use of soils and gravels. Since they had only the slightest understanding of the mechanics of materials or of flood flow, their methods were haphazard, and their works often failed. Embankment dams were low on the scale of public confidence for many centuries."*

Today, embankment dams exist in excess of 300 meters high with volumes of many millions of cubic meters of fill. Thousands of embankment dams exceeding 20 meters in height have been constructed throughout the world. Currently, China is the leader in embankment dam construction.

The embankment dam is popular because:

- Materials available within short haul distances are used,
- The embankment dam can accommodate a variety of foundation conditions, and
- Often, the embankment dam is least costly when compared to other dam types.

However, before determining whether an existing dam is adequately designed or a proposed embankment dam is suitable for the dam site, the evaluation should investigate such questions as:

- Are the dam and its foundation susceptible to internal or external erosion?
- Is the dam subject to overtopping considering its operational characteristics and various credible loading conditions?
- Is structural sliding of the existing or proposed dam and abutment slopes a possible failure mechanism and, if so, is there an adequate factor of safety?

1.2. BASIC REQUIREMENTS OF THE EMBANKMENT DAM

Satisfactory performance of embankment dams must include the following:

- The embankment, foundation, and abutments must be stable against slumping, sliding and sloughing during construction, during all conditions of reservoir operation and during and following unusual events such as earthquake and flood.
- Seepage through the embankment, foundation, and abutments must be controlled and collected to prevent excessive uplift pressures, piping, sloughing, dissolution and erosion of material into cracks, joints and cavities. Because of low yield within the watershed, some reservoirs require a limitation on the rate of seepage. Foundation cutoffs, select core material, upstream impervious blankets, chimney filter and drain systems, blanket drains, finger drains, toe drains, multiple transition filters between core and rockfill shell material, drainage adits and tunnels, drain holes and relief wells are common measures to control and limit seepage. Redundancy and multiple defenses are often necessary and represent sound engineering practice considering the uncertainties at any given dam site. Existing dams that do not incorporate typical seepage defense measures may require prompt defensive action should a problem develop.
- Freeboard must be sufficient to prevent overtopping by wave action. An allowance for post-construction settlement of the dam and its foundation, and deformation caused by earthquake must be included. In addition, freeboard must be sufficient to pass the maximum design flood, often chosen as the probable maximum flood. Spillways and outlets must be designed with sufficient capacity such that overtopping of the dam does not occur.
- Outer slope protection on both the upstream and downstream slopes must prevent erosion by wave action, reservoir level fluctuations, rainfall and wind. Materials must be durable and resistant to wet/dry and freeze/thaw cycles. Materials must resist weather and erosion over long periods of time.
- The foundations must be properly prepared and treated during construction. Unsuitable material must be removed, water entering the foundation must be controlled, and foundation surfaces must be prepared to receive the first lifts of fill material. If the foundation is a rock surface, the treatment below the core will, at a minimum, include detail cleaning of the rock surface using air and, possibly, water and the application of slush grout and dental concrete, if required. The first few lifts of core material should be as plastic as possible and specially treated to

ensure a good bond with the rock foundation.

- The dam must be constructed using appropriate quality control and quality assurance procedures. Appropriate changes to the design must be made during construction should site conditions so indicate. The ultimate performance of the dam depends on careful construction especially regarding foundation treatment, moisture and density control of the fill, and the design and construction of filters and drains.
- During reservoir filling and project operation, routine inspections of the dam and its foundation and the evaluation of instrumentation data to identify abnormal behavior and the necessity for remedial treatment are required. Long-term acceptable performance will be assured by early recognition of problems and prompt remedial treatment. Danger signs include:

  - Erosion of the outer slopes, or of the abutments
  - Wet or saturated areas along the downstream slope
  - Seepage emerging on the downstream slope or from abutments and foundations
  - Changes in seepage rate or in the pore pressure distribution within the dam
  - Clogged drains, or seepage by-passing the drainage system
  - Seepage carrying fines
  - Cracks on the crest, the outer slopes, or within the abutments
  - Sink-holes or unexplained depressions
  - Increased settlement with time

1.3. EMBANKMENT DAM FAILURES

A variety of texts and publications discuss dam safety, the reasons for failures and accidents, and lessons to be learned. A review of the data from the 1975 and 1988 ASCE/USCOLD studies indicates that about 40 percent of failures and accidents to embankment dams are the result of leakage and piping through the dam, foundation, and/or the abutments. Flood discharge and/or overtopping and washout of the dam are a second major cause of failures and accidents. Slides within the abutments or the embankment slopes caused by a high phreatic surface within the downstream slope, drawdown of the reservoir, or earthquake are another major cause of failures and accidents to embankment dams.

Ralph Peck, in his Laurits Bjerrum Memorial Lecture, 1980, commented on the above. The following excerpts are from this lecture.

> *“We can infer ... that a failure is seldom the consequence of a single shortcoming. Usually there is at least one other defect or deficiency, and the failure occurs where two or more coincide. This inference supports the principle of designing to provide defense in depth, the 'belt and suspenders' principle long advocated by Arthur Casagrande. It postulates that if any defensive element in the dam or its foundation should fail to serve its function, there must be one or more additional defensive measures to take its place…*

"The bedrock treatment appropriate to the geological conditions is a matter of design. It is not an aspect of design susceptible, however, to numerical analysis. Instead, it requires the exercise of judgment, a sense of proportion. When a jointed bedrock foundation is being treated and covered with the first layers of fill - a crucial time with respect to the future performance of the dam - engineers fully acquainted with the design requirement should be present, should have the authority to make decisions on the spot, and should not delegate their authority unless and until they are satisfied that their judgment concerning the particular project has been fully appreciated by their subordinates.

"I doubt if guidelines, regulations, or even the best of specifications can take the place of personal interaction between designers and field forces at this stage...

"The literature already has much to say about cracking of earth dams. The emphasis, however, is on the mechanics of producing the initial cracks, an aspect that has recently become at least partly amenable to analysis. The analytical results serve a useful purpose: reduction of cracking can undoubtedly be achieved most successfully if the causes of cracking are understood and avoided. Nevertheless, to accord with the principle of defense in depth, every dam should be designed on the assumption that the core may crack and that the dam should be safe even if it does.

"So we must reckon with the conclusion that modern dams seldom if ever fail because of incorrect or inadequate numerical analyses. They fail because inadequate judgment is brought to bear on the problems that, whether anticipated or not, arise in such places as the foundation or the interface between embankment and foundation."

1.4. UNDERLYING CONCEPTS

The theme of this chapter and of this report is the satisfactory performance of the embankment dam through appropriate selection and understanding of materials. This satisfactory performance must be achieved throughout the useful life of the dam and reservoir, a period of time that could span hundreds of years. To achieve this, the following guiding principles are suggested:

1. **Design defensively, using redundant systems.** For example, a well designed and constructed core, facing or internal membrane backed up by appropriate filters, drains and transitions with sufficient capacity to safely accept flow from cracks or other defects. The many failures and accidents caused directly or indirectly by leakage and piping within the dam, the foundation or the abutments point to the necessity of multiple lines of defense.
2. **Use experience and conservative judgment in selecting foundation preparation and treatment procedures**. The only appropriate opportunity to treat the foundation is when it is exposed during construction. It is difficult, expensive, and sometimes impossible to further treat the foundation after much of the embankment has been placed or after the reservoir has filled.
3. **Continually review and change, if necessary, the "design" of the dam**. This

process starts when the first reconnaissance of the site occurs and it continues through detailed site investigations, through design and analysis of the dam and its foundation treatment, through construction, and during reservoir filling and project operation. The owner of the dam must understand that it is not possible to eliminate all uncertainties that could affect construction and the final cost. The design of the dam is modified as the design process proceeds through site investigations and the analysis of data, and evolves as a better understanding of material and foundation properties is obtained. During construction, foundations are exposed and treated, and borrow areas are opened, yielding data which was not available earlier. The design is challenged and changes are made as needed.

4. **Seek peer review throughout the planning, design and analysis, construction, and operation of the dam and reservoir**. In the United States, for major dams and/or unusual site conditions, the Federal Energy Regulatory Commission requires an independent board of consultants to advise the owner with respect to the hydrologic and structural safety of the dam and reservoir from the start of planning studies to project start-up and operation. State dam safety agencies require compliance with specific standards for design and construction. Commonly, on major international projects, an independent panel of experts meets periodically to provide experience and judgment concerning critical design and construction issues regarding foundation treatment, materials, and lines of defense.
5. **Throughout the life of the project, evaluate the performance of the dam and reservoir using visual observations and instruments**. Detailed inspections, conducted regularly by walking the crest, slopes, toe and abutments of the dam, provide a visual record of performance. Evaluation of instruments that measure water pressure, seepage rate and deformation provides additional insights concerning performance. Frequent inspections and data evaluation provide the means to judge the performance and structural health of the dam and its foundation.
6. **Undertake remedial treatment promptly and in advance of a serious incident.** Any abnormal performance of the dam, the foundation or the abutments, as observed during the visual inspections or as a result of data analysis, must be evaluated to determine the potential impact to the safety of the dam. If it is determined that the safety of the dam may be compromised, the design and construction of remedial repairs should be undertaken immediately.

1.5 REFERENCES

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CHAPTER 2 — SOIL MATERIALS

2.1. INTRODUCTION

Embankment dams are constructed of all types of geologic materials, with the exception of organic soils and peats. Most embankments are designed to utilize the economically available on-site materials for the bulk of construction. Special zones such as filters, drains and riprap, may come from off-site sources. Soil materials used in embankment dams commonly are obtained by mass production from local borrow pits, and from required excavations where suitable.

This chapter deals with soil materials and their engineering characteristics that are significant to embankment dam design. Inorganic soils generally are divided into two broad categories for engineering purposes: fine grained soils and coarse-grained soils. Many embankments are constructed of broadly graded soils which do not fit entirely into either category, and exhibit characteristics of both. Moraine deposits are an important example of these mixed soils. The intent of this chapter is to provide a general overview of soil materials used in embankment dams. General construction considerations relevant to soils in embankment dams also are reviewed.

2.2 FINE GRAINED SOILS (SILTS AND CLAYS)

Fine grained soils are often used in homogeneous dams, and in impervious core sections of zoned embankment dams. The general characteristics, properties and uses of fine grained soils in embankment dams are described in this chapter. Special concerns are briefly discussed regarding the use of low strength and compressible materials, and dispersive clays.

2.2.1 General Characteristics and Index Properties

Fine grained soils are defined as materials having at least 50 percent by weight of particles finer than 0.074 mm, or the openings of a U.S. Standard No. 200 sieve, according to the Unified Soil Classification System (USCS). Inorganic, fine grained soils are classified as either clays or silts. Clays and silts typically exhibit different engineering behavior with regard to compressibility, strength and permeability. Clays and silts are separated, somewhat arbitrarily, on the basis of the 2 micron (0.002 mm) size fraction, with clay particles being defined as smaller than 2 microns, and silt sizes between 2 microns and the No. 200 sieve size. Fundamentally, fine grained soil behavior is determined by such things as the shape of the microscopic particles, clay mineralogy and crystal chemistry, nature of interparticle bonding, soil fabric and soil pore water chemistry, in addition to particle size. These microscale, physio-chemical characteristics are not determined routinely for engineering purposes. Geotechnical engineers instead use index properties, primarily soil plasticity and gradation (particle size distribution), to classify both fine grained and coarse grained soils, and to indicate potential soil behavior. Index properties are measured by simple, standardized laboratory tests.

The most common index tests, the Atterberg limits, help geotechnical engineers classify and characterize fine grained soils according to the degree to which moisture changes impact soil consistency and behavior. Atterberg limits are water contents at defined transitions in soil consistency, as measured by standardized tests. The liquid limit and the plastic limit are the most commonly used Atterberg limits in engineering work. The liquid limit is the water content at the transition between a liquid and plastic solid state, and the plastic limit is the water content that defines the lower limit of the plastic solid state. The plasticity index (PI = liquid limit - plastic limit) indicates the magnitude of water content range over which the soil remains plastic. The USCS includes a plasticity chart which allows classification of fine grained soils on the basis of liquid limit and PI. The USCS soil classification procedure is described in most soil mechanics texts, and in ASTM D2487 (ASTM, 1998, or latest version). In European practice, notably in France, the methylene blue test is also used to complement consistency limits testing.

The principal characteristics that distinguish fine grained soils from coarse grained soils for purposes of embankment dam design are that fine grained soils have lower permeability, lower shear strength, and higher compressibility. Soil plasticity serves as an initial indicator of the potential behavior of a clay or silt when placed in an embankment dam. Consider, for example, a clay and a silt having the same liquid limit. The clayey soil will be more plastic, that is, it will have a higher PI. It likely will be less permeable than the silty soil. The compressibility of both soils may be nearly the same. If two soils having the same PI are considered, the soil with the higher liquid limit will be more compressible, whether that soil is a silt or a clay.

Index characteristics are used appropriately as classification tools, but not for design. Index tests are made on thoroughly disturbed soil specimens that have been screened and reworked. As such, the natural soil fabric is destroyed. This fabric is especially important in understanding the behavior of tropical residual soils and *in situ* loess. Laboratory tests on representative intact foundation specimens, or remolded borrow material specimens, are needed to adequately estimate the important design properties of permeability, strength, and compressibility of the fine grained soil materials that are present in the foundation or will be used in the dam fill. However, index properties, including Atterberg limits and gradation, are often used in specifications to control the types of materials that are placed in various embankment fill and structural backfill zones.

2.2.2 Shear Strength

There is a strong correlation between incidence of embankment slope failures and the use of fine grained/highly plastic soil in embankments. Excess pore pressures often develop during rapid construction of fine-grained fill zones, resulting in reduced shear strength and potentially unstable conditions during or shortly following construction. Early studies done by Sherard (1953) indicated that the correlation between fineness of soil and the susceptibility to sliding was strong enough to outweigh the influence of all other factors, including steepness of slopes, construction methods, and reservoir activity. The USBR recommends shallow slopes (3H:1V to 4H:1V for upstream slopes, and 2.5H:1V for downstream slopes) for homogeneous or modified-homogeneous small dams constructed of fine grained soils. For reasons of safety and economy, a zoned embankment consisting of a central or sloping impervious core flanked by zones of higher-strength pervious materials, should always be constructed in areas where there is a variety of soils available (USBR, 1987).

2.2.3 Compressibility

Compressibility of embankments depends on the soil properties and the placement conditions. Under lower consolidation stresses, such as in the upper sections of an embankment, compressibility appears to correspond with placement moisture. Fill placed at relatively low average water contents shows low initial strain and moderately increasing or constant compressibility under higher pressures. Some materials may exhibit collapse settlement on wetting if placed at low initial moisture. Embankments may exhibit high initial strains when constructed at water contents near standard Proctor optimum, as determined by ASTM D698 (ASTM, 1998, or latest version). The potential for collapse settlement, or high initial settlement can be predicted on the basis of laboratory testing prior to construction. Gradation and plasticity of embankment soils are considered to be more important than placement moisture conditions under high consolidation pressures, such as in the lower portions of a high dam (Sherard, et al., 1963).

Differential settlement is a particular concern. Differential settlement will be most severe at steep abutments where there are large differences in embankment height over short lateral distances, or near internal structures where adequate compaction is difficult to achieve. Shaping of abutments should be done to provide smooth changes in embankment height. Foundation treatments should include trimming and/or placement of dental concrete to eliminate sharp differences, or large steps in rock foundations. The United States Bureau of Reclamation (USBR) provides guidelines for these treatments (USBR, 1984). Fill placed against small irregularities in rock foundations or concrete surfaces must be sufficiently plastic to accommodate differential movement across the discontinuity without cracking (Jansen, 1988). This is often achieved by selecting materials with high plasticity, and compacting the materials wet of optimum moisture in these critical areas.

2.2.4 Earthquake Resistance

Homogeneous dams constructed of clayey materials have been proven to be highly resistant to earthquake damage. During the great California earthquake of 1906, with an estimated Richter magnitude of 8.3, about 30 medium-sized embankment dams in the immediate vicinity of the fault rupture were subjected to strong ground motions. Most of those dams had a homogeneous section of clayey fill. None of the embankments failed during the earthquake, and most survived the shaking with minimum damage. In fact, embankment dams built of compacted clayey materials have historically withstood extremely strong levels of ground motion, even when obsolete or inefficient compaction procedures were used to construct them (USCOLD, 2000, 1999 and 1992).

2.2.5 Dispersive Clays

Dispersive clays have unique physio-chemical characteristics that make them potentially highly erodible under even low hydraulic gradients, as compared to non-dispersive clays. A detailed explanation of these characteristics is provided in ICOLD (1990b). Internal erosion of dispersive clays that may lead to piping failure of the dam is usually initiated in areas where seepage forces are moderate to high, such as in areas adjacent to soils having high hydraulic conductivity, around conduits, against concrete structures, at foundation and abutment interfaces and along cracks in the core. Surface erosion of dispersive soils is a consideration when these soils are used on the outer slopes of the embankment, or comprise foundation or abutment materials.

The tendency for dispersive erosion hinges on clay mineralogy and soil water chemistry. The principal difference between dispersive clays and ordinary erosion-resistant clays is that dispersive clays have a high percentage of sodium cations adsorbed on the clay particle surfaces, relative to other common soil cations such as calcium, magnesium and potassium. When these clays come in contact with water low in dissolved salts the particles tend to disperse or deflocculate, and can then be easily carried away by flowing water.

The possible existence of dispersive soils always should be considered in any geotechnical investigation, especially where borrow sources are derived from alluvial, loessial, or marine deposits, or where there is surface evidence of these soils. Surface evidence includes unusual erosion patterns with tunnels and deep gullies, together with excessively turbid water in impoundments. Dispersive clays can be identified by a number of field and laboratory tests, which are described in ICOLD (1990b).

When dispersive soils are identified, but economic factors require use of these materials in an embankment, special provisions must be employed in design of the dam. Of these the most important is design of proper filters. Almost all of the considerable number of failures attributed to dispersive clays have occurred in homogeneous dams without filters, and all dispersive piping failures were caused by the occurrence of an initial concentrated seepage path through the embankment. Use of adequate filters should preclude these types of failures. Compaction and moisture control also are critical. Special care should be taken to ensure that proper compaction is achieved around conduits and other structures, and at steep abutments and foundation contacts. ICOLD (1990b) provides a detailed summary of dispersive soils in embankment dams, including measures which can be used to prevent problems associated with these materials.

2.3 COARSE GRAINED SOILS (SANDS AND GRAVELS)

Coarse grained soils are used in structural fill zones, or shells, and in specialty filter and drain zones within embankment dams. Coarse grained soils are also used in core zones, especially when the fines content is greater than 20 percent. The general physical characteristics and properties of coarse grained soils are briefly described in this section.

Special concerns regarding the permeability, erosion potential, and liquefaction potential of coarse grained soils are briefly discussed.

2.3.1 General Characteristics and Properties

Coarse grained soils are defined by the USCS as those materials having more than 50 percent by dry weight of particles retained on the U.S. Standard No. 200 sieve, or 0.074 mm. Coarse grained soils include gravels and sands, which are distinguished rather arbitrarily by size. Sands are defined as soils finer than the No. 4 sieve (4.76 mm) and coarser than the No. 200 sieve. Gravels are coarser than the No. 4 sieve and finer than 3 inches (76.2 mm). This size division does not correspond with a distinct change in engineering behavior, although in general, gravels are more pervious and exhibit greater shear strengths than sands.

Clean sands and gravels, meaning sands and gravels that have less than about 5 percent fines by dry weight are pervious, easy to compact, and are minimally affected by changes in moisture. The important properties of interest in embankment dam engineering, namely shear strength, compressibility, and permeability are determined by the gradation, grain size and shape, relative density, and durability of the coarse grained soil. Compressibility is generally of less concern, as these soils are essentially incompressible when compacted to a dense state.

2.3.2 Hydraulic Conductivity (Permeability)[1]

Clean gravels have high hydraulic conductivities, ranging on the order of 1 to 100 cm/s. The hydraulic conductivity, k, of clean sands or clean gravel-sand mixtures can be reasonably estimated based on semi-empirical correlations with *effective grain size, D_{10}*. The Hazen equation for example, can be used to estimate k in cm/s using the relationship

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

where, D_{10} = grain size in mm corresponding to 10 percent passing on the gradation curve for the soil. This equation is considered valid for materials having D_{10} sizes between 0.1 and 3.0 mm. The constant C varies between 0.4 and 1.2, with an average value of 1 (Holtz and Kovacs, 1981). Lambe and Whitman (1969) provide a summary of permeability values and D_{10} size correlations for a variety of soils. Sherard, et al. (1984) related hydraulic conductivity to the D_{15} size for uniform filter sands according to:

$$k = C(D_{15})^2$$

where, C = 0.2 to 0.6 with a mean of 0.35. Shepherd (1989) summarized a number of similar empirical correlations for a variety of coarse grained soils.

[1] Geotechnical engineers often use the terms *permeability* or *coefficient of permeability* instead of hydraulic conductivity. In ground-water and geoenvironmental disciplines it has become customary to use the term hydraulic conductivity (in L/T units) when referring to the proportionality constant relating flow velocity and hydraulic head, and *intrinsic permeability* (in L^2 units) when referring to the material property.

Hydraulic conductivity is measured in the laboratory using standardized procedures on representative samples. The USBR measured hydraulic conductivities of well-graded sand and gravel mixtures in the range of about 1 to 5 X 10^{-2} cm/s, for mixtures containing 20 to 65 percent gravel sized particles at relative densities ranging from 50 to 70 percent. At higher or lower gravel percentages, hydraulic conductivities were found to be substantially higher (USBR, 1974, 1990).

It is imperative that pervious zones remain pervious throughout the life of an embankment. Filters and drains may become clogged by migration of clayey fines resulting from gradual alteration of originally cohesionless materials, or by precipitation of chemicals. Materials that tend to weather, break down under compaction, or may be susceptible to recementation should be avoided in critical zones such as filters and drains. Additional discussion of filter and drain zones is provided in Chapter 4.

2.3.3 Relative Density

Shear strength is directly proportional and hydraulic conductivity is inversely proportional to relative density of granular materials. Relative density is defined as

$$D_r = \frac{e_{\max}\ e}{e_{\max}\ e_{\min}}$$

where, e_{max}, e_{min}, and e = maximum, minimum and in place void ratios, respectively. The maximum and minimum void ratio are defined, and evaluated, by standardized laboratory testing procedures (ASTM Methods D 4253 and D 4254, respectively). Typically, relative densities are specified to be on the order of 75 percent or greater in structural fill. High relative density correlates with high strength, resistance to liquefaction under earthquake shaking, and reduction of risk for settlement upon saturation.

2.3.4 Surface Erosion Potential

Clean sands and fine gravels tend to be highly vulnerable to surface erosion under wave action and surface runoff. These materials generally are not used on the outer slopes of embankments. Erodible materials are protected by properly bedded riprap, soil cement, or other revetments on the upstream slope, and by coarse gravel, cobble or rock blankets, or by proper, shallow-rooting vegetation on downstream slopes.

2.3.5 Liquefaction Potential

Liquefaction is a term that has been applied to different, but overlapping, phenomena that occur in loose sands and gravels subjected to cyclic loading. One phenomenon is slope failure caused by loss of shear strength during undrained shear of highly contractive, fully saturated zones. If the soil mass liquefies along a critical failure surface, and is unrestrained, the mass appears to flow when this type of catastrophic failure occurs. For this reason the phenomenon is referred to as a flow slide. The other phenomenon is cyclic deformation. Deformations caused by cyclic loading may or may not lead to failure of the dam. Failure could occur if the embankment settles to a point where the normal pool overtops the dam. Also, failure may occur by uncontrolled seepage if large deformations affect the integrity of the zoned sections of the dam or open cracks in the embankment fill.

Once the seismic loading of the dam is determined, the general design approach is to assess if liquefaction may be triggered. Generally if liquefaction is triggered, the pore water pressures increase until the strength of the material drops to some residual value. The most recent publication on evaluating liquefaction resistance of soils is NCEER (1997). Even if a flow slide caused by liquefaction is not predicted, significant strength loss may occur in the embankment materials. Using the appropriate reduced strengths for various zones of the dam, the amount of deformation along critical failure surfaces is estimated.

Loose, saturated sands are most vulnerable to liquefaction. Sands exhibiting low blow counts (from Standard Penetration Tests or Cone Penetration Tests done in borings), uniform gradations, and rounded grains are likely to be potentially liquefiable. The determination of liquefaction potential is independent of the stability analysis. The stability analysis is used to determine the effect of liquefaction, and whether the driving shear loads in the dam under the cyclic shaking are less than or greater than the available shearing resistance.

The potential to trigger liquefaction can be greatly reduced by constructing dense, well-compacted embankments on competent foundations. Other methods to reduce or mitigate the effects if liquefaction is triggered include:

- reducing the embankment slopes,
- adding berms at the toes,
- lowering the phreatic line or desaturating the critical zones,
- stone columns, and
- removal and replacement of vulnerable materials.

Tailings dams, and some older dams constructed of sands and silty sands using hydraulic fill construction methods, are by far the most vulnerable to earthquakes. Older dams built of inadequately compacted sands or silts, and tailings dams represent nearly all the known cases of failures under earthquake loadings, primarily as a result of liquefaction of the embankment materials (USCOLD, 1992). Hydraulic fill techniques are rarely used in modern dam construction because of this concern.

Cyclic deformation can occur even in soils that are not susceptible to complete liquefaction. Deformation of embankments during seismic events should be evaluated if the factor of safety against liquefaction is less than 1.5, and if the design peak ground acceleration exceeds about 0.15*g* (15 percent of gravitational acceleration), according to S.J. Poulos *in* Jansen (1988). A deformation analyses procedure for embankment dams is presented in Makdisi and Seed (1978). Marcuson et al. (1992) provide a review of seismic stability and deformation analyses procedures.

2.4 BROADLY GRADED SOILS

2.4.1 General Characteristics and Properties

Many natural soil deposits comprise a large range of particle sizes, and their engineering behavior is intermediate between fine grained and coarse grained soils. Broadly graded soils typically exhibit properties of low hydraulic conductivity, high shear strength, and low compressibility in comparison with fine grained soils. These engineering properties are related to their broadly graded particle size distribution.

Moraine, or till[2], is an important example of broadly graded soil materials because moraines cover large areas in the northern hemisphere. Table 2.1 summarizes some of the engineering material properties of morainic soils used in embankment dams. Colluvial and bouldery alluvial deposits are also significant sources of broadly graded soils that are used in embankment dams.

2.4.2 Material Characterization

The primary difficultly associated with use of broadly graded soils relates to the potential for wide variation in homogeneity of a particular deposit. Moraine formed by glaciers in particular can be highly heterogeneous because of "contamination" by lateral debris falling from valley side walls, and particle sorting by recurrent melting periods in the glacial history. Geotechnical investigations and characterizations of moraine sites are sometimes very difficult and require special attention. Obtaining or reproducing representative laboratory samples for testing of gradation, permeability, strength and compressibility is often a formidable challenge. Deep exploratory trenches are better than boreholes for examining homogeneity, and for acquiring representative large size samples.

[2]The term *till* is often used as a synonym for moraine, especially in Europe.

Table 2.1. Engineering Properties of Moraine Materials Used in Embankment Dams (adapted from ICOLD, 1989)

PROPERTY	MIN. VALUE	MAX. VALUE	TYPICAL VALUE
Passing No. 200 (%) (United States)	20	71	
Passing No. 200 (%) (Scandinavia)	14	55	
Passing No. 200 (%) (Russia)	5	22	
PI (%) (Western/Central Canada)	3	27	
PI (%) (Other Areas)			NP
Optimum Water Content (%)	5	16	7-10
Shear Strength, φ (deg.) (Western/Central Canada)	23	37	
Shear Strength, φ (deg.) (Eastern Canada/Scandinavia)	35	45	
Hydraulic Conductivity (m/s)	10^{-11}	10^{-6}	

2.4.3 Internal Erosion and Piping

The use of broadly graded soil as the impervious element of a dam requires supplemental lines of defense to protect against internal erosion and piping. Core width, adequate filter zones and material selection and mixing are extremely important design issues for these types of materials.

Wide cores can also hydrofracture, but it is unlikely that the stress conditions remain favorable for cracking through the complete width of the core, and any crack would be only partial. Minimum core widths of 0.25 to 0.3 times the hydraulic head are considered appropriate when vertical broadly graded cores are used.

Selection of proper filter material and adequate width of the filter/transition zones are of paramount importance. Filter criteria for broadly graded, cohesionless soils must consider the internal stability of the base soil, and the self-filtration process taking place at the base soil/filter interface. An internally unstable soil has a structure that allows finer particles to migrate within its own coarse particle matrix. Gradation curves of internally unstable soils generally have an upwardly concave shape. The filtration process at the base soil/filter interface for these soils can lead to unavoidable reduction in hydraulic conductivity (Lafleur, et al., 1989). Special considerations for design of filters next to broadly graded materials are described in Chapter 4.

2.4.4 Earthquake-Induced Settlements

Embankments constructed of broadly graded soils are, in general, highly resistant to earthquake damage. However, it is believed that earthquakes could trigger sudden and large settlements in certain high embankments having cross sections with broadly graded cores and less compressible adjacent filter zones (ICOLD, 1989). This is caused by arching phenomena which develop when there is significant differential settlement between core and filter zones after construction. Zones of less consolidated material may exist beneath the "hanging" core sections. These less consolidated materials settle during seismic excitation, collapsing the overlying arched zones. This phenomenon may also occur in dams having fine-grained cores. Wide core zones mitigate this phenomenon.

2.5 DESIGN AND CONSTRUCTION CONSIDERATIONS

2.5.1 Fine Grained Soils

Fine grained soils typically are used as the water barrier in embankment dams, either in a homogeneous section, or as the core in a zoned embankment. The U.S. Bureau of Reclamation's *Earth Manual* (USBR, 1974, 1990) identifies the following criteria for design of impervious earthfill zones:

- The material must be formed into an essentially homogeneous mass, free from any potential paths of percolation through the zone or along the contacts with the abutments or concrete structures.
- The soil mass must be sufficiently impervious to preclude excessive water loss through the dam.
- The material must not consolidate excessively under the weight of superimposed embankments.
- The soil must develop and maintain its maximum practicable shear strength.
- The material must not consolidate or soften excessively on saturation by water from the reservoir.

Historically it was believed that core materials should be constructed of clay rather than silt. This belief was popular because clay is less permeable than silt, and clays were considered to be less vulnerable to internal erosion under concentrated leaks. Current practice tends to place more emphasis on utilizing economically available resources, including silty materials properly moisture conditioned and compacted. Design practice also emphasizes internal filter and drain zones and transition zones as needed for gradation change between the core and shell zones. These transition zones provide drainage protection against piping and serve as "crack stoppers" for the more brittle silty core material. It is preferable to avoid brittle cores and use more plastic materials, if available. Placing silty material at a higher moisture content mitigates the brittleness at the expense of added construction pore pressure, and settlement. Dam design requires making the best use of engineering characteristics of the various materials available for construction of the embankment.

The size and shape of the impervious core in a zoned dam will depend on the availability of materials and their properties, especially hydraulic conductivity. Site conditions requiring specific construction sequencing may also be a factor in the zoning design. Cores typically are centrally located within the embankment or located upstream and sloped. A general rule is that the base width of the core be at least 25 percent of the maximum head (Jansen, 1988). Higher plasticity materials allow thinner cores. The potential for hydraulic fracturing must be considered in thin core designs because of differential settlements between the core and shell zones leading to arching and stress reduction within the core.

The design objectives for impervious embankment zones include minimizing permeability and consolidation, and maximizing resistance to softening on saturation. These goals are best achieved by maximizing placement density under carefully controlled moisture conditions. Maximum density is achieved at optimum moisture content for a given compaction energy. However, other factors may influence the specified placement moisture. Pore pressure development and formation of slickensided layers may be minimized during placement of embankment core materials if the soil is placed at moisture contents slightly below optimum moisture. However, placing at less than optimum moisture can result in deformation on saturation. Climate and in situ borrow conditions may also be important factors. If the borrow source materials have a natural moisture content wet of optimum, and the climate is humid, reducing the moisture poses considerable difficulty and may produce undesirable effects related to shrinkage and cracking. It is common practice under these circumstances to compact core materials at moisture contents above optimum (ICOLD, 1990a).

High pore water pressures may develop during construction of fine grained embankment sections because underlying materials consolidate under the weight of added fill. These pressures may exceed any that will occur in the subsequent lifetime of the embankment. Sherard, et al. (1963) listed a number of design and construction procedures to minimize construction pore pressure development, and eliminate the need to control design on the basis of these relatively short-term conditions. These procedures are listed below, with USSD Committee on Materials for Embankment Dams editorial comments noted in italics to indicate limitations that have since been recognized:

1. Compact the impervious section of embankment at an average water content a few percent below Standard Proctor optimum.
   [*Committee note: Compaction significantly below optimum moisture presents a risk of post-construction collapse settlement of compacted fine-grained materials. This potential can be tested in the laboratory.*]
2. Make the impervious section thinner so that high construction pore pressures will have less influence on the stability and will dissipate more rapidly.
   [*Committee note: The use of thin cores has its own risks, including higher exit gradients at the core/filter zone interface, greater potential for differential settlement cracking extending across the core, and constructability problems.*]
3. Install internal drains within the impervious section [*of a homogeneous dam*] to accelerate the pore pressure dissipation.

4. Control rate of construction to allow more time for pore pressure dissipation. [*Committee note: Sometimes construction pore pressures do not dissipate before reservoir filling, and overall stability may be compromised*.]
5. Lower factors of safety can be tolerated against the possibility of slope failure during construction than would be permissible under long-term loading conditions under reservoir head.

Construction methods and equipment, material properties, and environmental conditions must all be considered in planning the control of placement of embankment core materials. Test embankments are often constructed to determine optimum placement conditions and procedures including placement moisture, lift thickness, type of compaction equipment and number of coverages per lift. Test embankments are always good practice, and often can be incorporated into the permanent embankment section.

Inclusion of high percentages (on the order of 30 percent or higher) of gravel or cobble size particles in otherwise fine-grained materials can have a significant impact on engineering properties, especially shear strength. Inclusion of coarse material changes optimum moisture contents, and compaction energy requirements. Compaction may be impaired by the presence of large particles, and often an upper size limit is established to ensure proper compaction is obtained. USCOLD (1988) provides procedures for construction testing of embankment materials containing large particles.

2.5.2 Coarse Grained Soils

Gravels and sands typically are used in the shells or in transition zones of zoned embankments, and in filters and drains. Gravels and sandy gravels are sometimes used as the primary section of an embankment with an upstream facing of asphaltic concrete or Portland cement concrete.

Shell and transition zones are intended to provide strength and support for the impervious core, to ensure good drainage, and to act as filters between zones of differing grain size. Shell materials must exhibit adequate shear strength at economical slopes. Upstream shells should be as free draining as possible to ensure stability during rapid reservoir drawdown and under earthquake loadings. Poorer quality shell materials, i.e., materials containing more fines or which may break down on placement or exposure to elements and end up less pervious, may be used in downstream sections if adequately filtered internal drainage systems are provided (Jansen, 1988).

To ensure proper drainage, the USBR recommends the ratio of hydraulic conductivities of permeable zone to impermeable zone be at least 10, and preferably much larger. The hydraulic conductivity of the free-draining zone should be sufficiently high to preclude development of pore pressures during construction.

The use of sands and gravels in filter and drain zones is described in detail in Chapter 4 of this document.

The U.S. Bureau of Reclamation's *Earth Manual* (USBR, 1974, 1990) identifies the following criteria for design of pervious earthfill zones:

- The material must be formed into a homogeneous mass free from large voids.
- The soil mass must be free draining.
- The material must not consolidate excessively under the weight of superimposed fill.
- The soil must have a high angle of internal friction [i.e., high shear strength].

These objectives are best achieved through careful selection and processing of materials for filter/drain zones, and control of placement and compaction for structural fill zones.

Good quality coarse material sources exist in the vicinity of many dam sites. However, specifications for filter and drain zones typically limit the percentage of fines, after compaction, and define gradation boundaries which are rarely met by the materials in their natural state. Processing is generally required, including washing to remove fines, handling to preclude size segregation, and remixing to achieve specified gradation requirements (ICOLD, 1990). Also, the quality of materials should be such that they do not break down into smaller sizes during placement, or weather excessively with time. Gradations should be checked before and after compaction of these materials to ensure that particle breakdown is not a problem.

Materials in the embankment shells, excluding filter and drain zones, should be compacted to maximum practicable densities. Heavy, vibratory compaction equipment generally works best. Moisture control is not as critical for gravels, gravelly sands, silty sands, and sandy gravels, as it is for fine grained materials. A small amount of added moisture enhances compaction of coarse grained soils with fines. Compaction control is done by various means. In-place densities of sands can be checked by conventional methods such as nuclear density probes, sand cone tests, or standard penetration tests. In gravels, these tests are not practical because of the large grain size relative to the test equipment. Compaction control in gravel zones is often accomplished by specifying a certain number of coverages of a specified type of equipment, and a maximum lift thickness. In place density can be checked on representative test fills, and occasional tests during construction, using the USBR Ring density test, which is now published as an ASTM standard (ASTM Method D5030). Construction testing of embankment materials containing large particles is presented in USCOLD (1988).

2.5.3 Broadly Graded Soils

Moraine deposits cover large areas of North America, Europe and Asia. These materials have been used extensively as fill for impervious cores in zoned embankment dams or for the main section of homogeneous embankments.

Most designers prefer using sandy to silty soils having broad gradations for construction of impervious cores or homogeneous dams because these materials tend to lose most of the excess pore pressures developed during placement at a substantially faster rate than clayey materials. Very low seepage rates are measured at most dams built with broadly graded cores. Post construction settlements are usually small and within acceptable limits. Embankments constructed of broadly graded soils are in general highly resistant to earthquake damage.

The design objectives for embankment zones comprising broadly graded materials are low hydraulic conductivity, low compressibility, high shear strength, and internal stability of the soil matrix. Placement and compaction procedures used to achieve these goals vary, but a variety of techniques have been effective. These include:

- Compaction in layers using vibrating compactors or pneumatic rollers — used mainly in North America to compact non-plastic soils
- Compaction in layers using sheepsfoot or padfoot rollers — used in western Canadian provinces to compact clayey moraine
- Wet compaction — used predominately in Scandinavia. This is a process where wet moraine is spread and compacted by one of two methods: (a) thin lifts compacted with heavy bulldozers, or (b) thick lifts compacted with vibrating rollers
- Dumping into water pools — used in (former) USSR. This method involves construction of segmented basins formed by diking along the core alignment. The basins are filled with water, and moraine dozed into the filled basins, and compacted by repeated passes of the dozers and haul trucks.

Moisture control of the borrow materials obviously depends on the placement procedure that is used. When broadly graded material is spread and compacted by conventional methods, moisture content is specified near optimum. Compaction of silty soil is more sensitive to water content variation, especially when water content exceeds optimum. When the wet compaction method is used, water content may be in the order of four to six percent above optimum. For moraine placed in pools, water content at the time of placement does not greatly influence final density, but higher placement water contents tend to reduce segregation during placing operations.

Density control is generally stringent for the conventional rolled fill placement approaches. A minimum of 97 to 98 percent standard Proctor dry density is typically specified. In locations subject to high earthquake loading, current practice is to use 97 to 98 percent of a higher compactive effort. For example, in California, USA, a 20,000 ft-lb/ft^3 (approximately 970 kN-m/m^3) compactive effort is typically specified, instead of the 12,400 ft-lb/ft^3 (600 kN-m/m^3) standard Proctor energy. For the wet placement process used in Scandinavia, 95 percent of modified Proctor (56,000 ft-lb/ft^3 , or 2,700 kN-m/m^3) is used. In Scandinavia, and particularly in the (former) USSR, the density of the material is not considered critical as long as the in place shear strength exceeds φ = 30E, and k # 10^{-6}m/s. Embankment design slopes reflect the anticipated shear strength.

Specifications controlling the materials and placement of soil materials in embankment dams often must be a compromise that will result in a proper balance among all the design criteria.

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CHAPTER 3 — ROCKFILL MATERIALS

3.1 INTRODUCTION

Rock has been widely used as a construction material for embankment dams. Until about 1940 it was used primarily as dumped rockfill in high lifts initially without sluicing with water, and later adding water sluicing to aid compaction. Compaction of rockfill material with mechanical equipment was started about 1960, and by 1980 most rockfill materials for embankment dams were well compacted in lifts not exceeding about 1.5 meters. This practice was initially started by track walking with heavy crawler tractors and later aided with heavy vibrating steel drum rollers. Currently, the rollers being used weigh about 90,000 kilograms.

Rockfill in current practice includes angular rock fragments as produced by quarry or occurring as talus deposits, and rounded or subangular fragments such as coarse gravel, cobbles, and boulders occurring in alluvial deposits. It is generally considered that the difference between clean rockfill and dirty rockfill is that in clean rockfill the rock content is sufficient to have rock to rock contact with the strength of the rock controlling the shear strength rather than the soils or fines. For many rock materials this occurs at rock content of 60 to 70 percent. Dirty rockfill with a hydraulic conductivity less than 1×10^{-3} cm/sec may be considered as earthfill because the possibility of developing construction pore pressures; more pervious material may be regarded as clean rockfill according to Penman (1976). The physical engineering properties of rockfill are difficult to evaluate and requires special testing procedures, particularly to determine the strength and permeability because of the large particle size. In many cases, the errors arising from an improper appraisal of the soil and rockfill material properties can far exceed those resulting from the use of more approximate methods of analysis, USBR (1987). Design criteria are not dealt with in detail in this report and sources such as ICOLD Bulletin 92, *Rock Materials for Rockfill Dams* (1993), with its numerous References should be utilized in detailed design. The majority of the material for this chapter is taken from ICOLD Bulletin 92.

3.2 REQUIREMENTS OF ROCK AS A CONSTRUCTION MATERIAL

There are two current trends in rockfill dam design. The first trend is to design rockfill dams with earthfill water barriers using the principle of fill-material zoning. Such dams are economical when all types of locally available rock of different strengths can be used in the structure zoning. The weaker rock is placed in less critical zones under less stress and hard sound rock is used where greater strength is required. Typical strength classification of rock unconfined compressive strength is listed below.

- High Compressive Strength 70 to 200 Mpa
- Medium Compressive Strength 17.0 to 70 Mpa
- Low Compressive Strength 3.5 to 17.0 Mpa

The second trend is to build rockfill dams with man made water barriers such as diaphragms or facings of reinforced concrete, asphalt concrete or other materials. Such dams are cost-effective by minimizing material volume for rockfill and water barrier features mainly through intense compaction of high strength rockfill materials with heavy vibratory rollers.

Currently rockfills are generally built of materials obtained from quarries in geologic formations or excavation of natural deposits of talus or alluvium. Explorations for rockfill materials formations rely on geologic mapping, core drilling, geophysical investigations and test blasts. Geologic mapping stresses rock type, jointing, weathering, overburden thickness and fracturing. The core drilling stresses the same items at depth and should provide representative coverage of the potential rockfill source. Seismic refraction surveys are commonly used in conjunction with drilling to assess quantity of the potential rockfill. P-wave velocities of more than 3,000 meters/sec indicates the rock is potentially a satisfactory source.

Test blasts and test fills provide considerable data on the suitability of the quarry and rockfill material. In addition the test blast and fill provide data on estimated construction costs. Test blasts and fills are expensive and are generally performed during final design or initial construction. They are expensive because of the large amount of construction work required to prepare the potential quarry for blasting and excavation for the test rockfills. The volume of rockfill needed for the test fill ranges from 2,000 to 8,000 cu meters depending on the number of rockfill types or zones being considered for the dam. The types of material that might be obtained are slope protection (riprap), pervious shell material (clean rockfill) and semi pervious shell material (dirty rockfill or random). Exploration of talus or alluvial deposits is performed in a similar manner except that test pits and trenches excavated with construction equipment are frequently used. The observation and sampling of the actual materials to be used, and test fills facilitates the evaluation of these sources.

All kinds of rocks (igneous, sedimentary and metamorphic) are used for compacted rockfill. In general, sound, intrusive, igneous rock (granite, syenite, diorite, gabbro, labradorite and so forth) features compressive strength of up to 250 Mpa at insitu specific weights of 2,700 kg/cum to 3,000 kg/cum. Porosity is low, less than one percent, as well as its water absorption, also less than one percent. Effusive rock, basalt, porphyrite, andesite trachyte, felsite and so forth, generally features lower compressive strengths, about 200 Mpa at insitu specific weights of 2,100 kg/cum to 2 950 kg/cum. Porosity of some material exceeds 1.0 percent.

Sedimentary rock, such as sandstone cemented by siliceous or ferrous materials, features strengths of 100 Mpa to 200 Mpa and 50 Mpa to 120 Mpa respectively. Strengths of sandstone cemented by calcareous and especially clayey materials are even lower, 60 Mpa to 70 Mpa or less. Water absorption of such rock ranges from 1.0 percent to 4.5 percent.

Strength, water absorption and other properties of metamorphic rock fall somewhere between those of igneous and sedimentary rocks; properties vary considerably depending on the degree of metamorphism and the composition of the host rock.

Intensely weathered and fractured igneous and metamorphic rock with reduced physio-mechanical indices is considered weak rock. Such rock is mainly pyroclastic (volcanic tuff, tuffite, tuffaceous rock), clastic (sandstone, conglomerate, siltstone), loamy (argillite, clay, shale) carbonaceous (limestone, dolomite, claymarl, chalk) and siliceous rock. Some of these materials when excavated, placed and compacted break down and produce a dirty rockfill embankment. Average engineering properties are as follows: insitu specific weight ranges from 1,800 kg/cum to 2,650 kg/cum; porosity ranges from 10 to 15 percent; coefficient of softening, (ratio between compressive strength of water-saturated and air-dry rock samples), ranges from 0.1 to 0.3 and compressive strength is less than 15 Mpa, sometimes even less than 5 Mpa. Lousnov (1981)

Laboratory tests are generally performed to determine specific rock engineering properties and are discussed in Section 3.3 below. Dam construction techniques (lift thickness, method of compaction and so forth) depend on the rock material used. Maximum rock size is conditioned by rock quality and dam-construction techniques, as described in Section 3.4, Rockfill Material Design and Construction Concerns.

3.3 DETERMINING ENGINEERING PROPERTIES OF ROCKFILL

The engineering and geological characteristics and physico-mechanical properties of rockfill materials are required for analytical validation of rockfill dam design. The most important properties of rockfill are as follows:

- Gradation
- Compacted Unit Weight
- Permeability
- Compressibility
- Strength and Deformation

3.3.1 Gradation

Rockfill properties are largely determined by the gradation and the strength of the rock particles. Figure 3.1 shows typical grading curves of several rockfill materials used in a modern large dam. Rockfill materials for compacted shell zones of modern dams are generally composed of coarse rock fragments or cobbles and gravel, ranging widely in particle size gradation, but generally with a maximum size of 18 to 48 inches, grading down to fines with 20 to 40 percent passing a 1 inch sieve and 5 to 15 percent passing a No. 4 sieve. Determining the potential gradation range of rockfill materials from quarry sites for large dams is best determined by test blasts and test fills. Gradation of rockfill materials for medium and small dams may be estimated from experience with the geological formation and core drilling. The large maximum particle sizes create unique problems in sampling, laboratory testing, design and handling.

Rockfill grain-size specifications have changed dramatically since the days when rockfill was dumped in thick lifts. Modern compacted rockfill contains a much higher percentage of fines. In fact, the more well-graded the material, the higher the unit weight of the placed material with the same compactive effort resulting in a less porous, denser embankment. Embankments of well-graded material have high moduli of deformation and there is less settlement as a result, as well as less crushing of the rock particles.

3.3.2 Compacted Unit Weight

Unit weight of compacted rockfill depends mainly on specific weight of the rock, grain-size distribution, compactive effort, lift thickness and compacting machinery. Compaction is achieved from the traffic of loaded trucks and spreading dozers supplemented by passes of a heavy vibratory roller or other compaction equipment. The first compacted rockfills (about the late 1950's and early 1960's) were compacted by track walking with heavy crawler tractors and with small vibratory rollers weighing about 3,000 kilograms, which were the largest available at the time. Later, 4,500 to 18,000 kilogram units became available, and although rollers of 11,000 to 18,000 kilograms were sometimes used, there is no evidence that units larger than 9,000 kilograms provided better compaction.

During the design phase, rockfill unit weight can be estimated from published data. During final design, rockfill unit weight and gradation can be confirmed from a test blast and a test fill. A test blast and test rockfill is not necessary for small and medium size dams when the compressive strength of the rock is medium to high. Observation of drill cores, saturated unconfined compression tests, published data and experience can be used to satisfactorily predict strength and compressibility ranges. Where rock is weak and saturated, and specimens show significant loss of strength, conservative placement specifications can be determined from past experience. A saturated test fill is advisable for large or high dams because many rock types have lower shear strength when saturated and loaded under very high confining pressures.

Unit weight in rockfill embankment is determined with standard test methods for density and unit weight of material in place with water displacement methods. The methods and special equipment needed to perform the tests because of the large particles are described in USCOLD (1988).

3.3.3 Compressibility

The characteristic of rockfill to decrease in volume under external load as a result of particle or fragment breakage, rearrangement, weight of overlaying materials and compaction is expressed as the modulus of compressibility. Dumped rockfill in high lifts has proved unsatisfactory for concrete-face rockfill dams because of its high compressibility. Faces were damaged and leaks developed in the lower parts of high concrete-face rockfill dams. Sluicing, to wash finer particles into voids and secure rock-to-rock contact, generally reduces compressibility. With rock of low compressive strength and high water absorption, sluicing with water may be important because loss of strength on saturation can be as much as 40 to 60 percent, Cooke (1990)

Moisture conditioning of dirty rockfill is generally required to ensure compaction of soil portion of the material.

Data obtained during construction using water-level settlement devices or crossarms have been used to determine representative moduli of compressibility for a variety of rock types placed with different procedures. Experience indicates moduli range from 27 Mpa to 128 Mpa, depending on the nature of the rock, the grading of the rockfill, lift thickness, compaction and other factors, Cooke (1990). Where the modulus is of particular concern, such as the upstream rockfill shell of a concrete-face dam, special placing procedures are specified to obtain maximum modulus.

3.3.4 Hydraulic Conductivity

The hydraulic conductivity of rockfill embankment is difficult to determine accurately because of the large particle size and segregation during construction. When securing representative samples for laboratory testing the maximum particle size of the sample is determined by the available laboratory hydraulic conductivity equipment. Testing of rockfill material with particles larger than about 6 inches is very difficult. Estimating a coefficient of hydraulic conductivity by using modeled gradation for testing is not very reliable. Fortunately, the exact coefficient of hydraulic conductivity is not needed for design purposes. Determination as to whether the rockfill is free draining, semi pervious or impervious is sufficient for design. These values can be determined by comparing gradations with those used in existing dams and field permeability tests in test fills, which is preferable. Rockfills with rock contents (plus 10 millimeters) of 60 to 70 percent can generally be considered free draining shell material.

3.3.5 Shear Strength and Deformation

Design Considerations

A thorough understanding and appraisal of the physical properties of soils and rockfill materials is essential to the use of current methods of design. No stability analysis, regardless of how intricate and theoretically exact it may be, can be useful for design if an incorrect estimation of the shearing strength of the construction materials has been made. In many cases, the errors arising from an improper appraisal of the soil and rockfill material properties can far exceed those resulting from the use of the more approximate methods of analysis. USBR (1987)

Current practice is to model the proposed rockfill with triaxial specimen 300mm to 400 mm in diameter using a maximum particle size of about 50 mm. Becker, Chan, Seed, Bolten (1972) has a good discussion of modeling rockfill materials for triaxial and plane strain testing. Selection of the gradation particularly the amount of fines, relative density, confining pressure, particle shape, and void ratio greatly affect the laboratory shear strength tests results for both plane strain and triaxial shear. For many dams, it is the practice to select shear strength values by comparison with test results obtained from published data for dams previously designed and constructed. Current stability analysis techniques allow for a variable shear strength dependent upon the confining pressure. Table 3.1 is a sample of shear strength under normal confining pressure for medium strength rock taken from ICOLD (1993).

Table 3.1. Shear Strength

	Triaxial Tests	
Normal Pressure (kPa)	Friction Angle ($Ø'_t$)*Degrees	Friction Angle ($Ø'_{ps}$)**Degrees
14	53	57
35	50.5	54
70	48.5	52
140	46.5	50
350	44	47.5
700	42	45.5
1400	39.5	43
3500	37.5	41

*After Leps (1970)

**ICOLD (1993)

Note: $Ø'_t$ = friction angle under triaxial deformation conditions.

$Ø_{ps}$ = friction angle under plane strain conditions.

Testing Equipment

Special equipment is required to determine rockfill strength and deformation parameters:

- Triaxial apparatus (compression and extension).
- Plane strain devices.
- Compression measuring apparatus (odometer).
- Direct shear apparatus.

The devices designed and built at the Instituto de Ingeniera, UNAM, Mexico Hirshfeld (1973), University of Tokyo Mogami (1977), and at the University of California Marachi (1969), are good examples of triaxial shear apparatus.

Triaxial apparatus can be used to determine rockfill strength and strain parameters. The UNAM triaxial apparatus was used to test cylindrical specimens 1.13 m in diameter and 2.5 m high at confining pressures up to 2,500 kN/m. The University of California triaxial apparatus was used to test specimens 0.9 m in diameter and 2.3 m high at a maximum confining pressures of 5,000 kN/m. The triaxial apparatus in University of Tokyo was used to test specimens 1.2 m in diameter and 2.4 m high. The testing at these facilities has been completed and the data obtained has been published. Large apparatus are not often used because equipment costs are high and large quantities of rockfill must be transported to the test facility.

3.3.6 Durability

Durability or resistance to weathering is usually of concern only for the surface layers of the embankment. The durability of rockfill materials is generally evaluated by laboratory testing of small samples of rock fragments to obtain index values so that comparison to existing satisfactorily performing rockfill can be made. Some of the common tests and acceptable values from ASTM (1993) are listed in the table below:

Table 3.2. Test Results for Evaluating Rockfill and Riprap

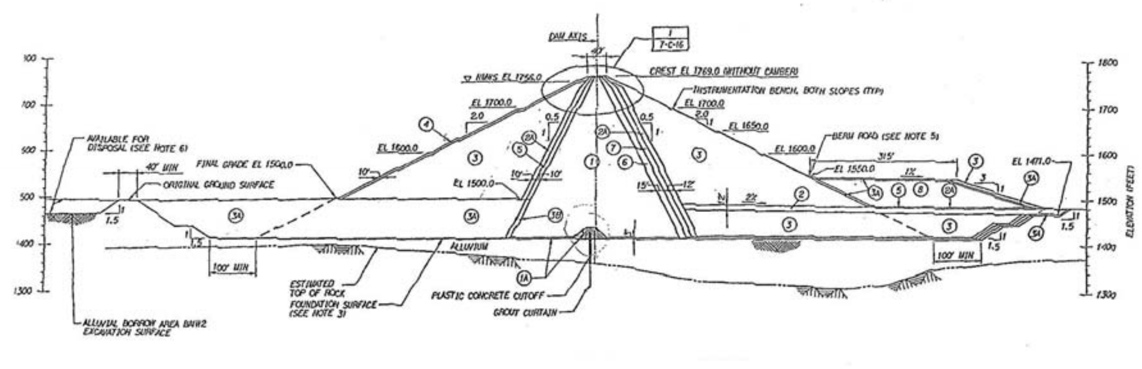
Characteristic	Range of 50 States Dept. of Transportation	Corps of Engineers
Soundness % loss		
Sodium Sulfate (max)	12-20	---
Magnesium Sulfate (max)	10-20	5
Ethylene Glycol (max)	---	0
Abrasion % loss (max)	40-60	20
Absorption % (max)	2-6	1
Density		
Specific Gravity (min)	2.3-2.5	---
Unit Weight g./cu. cm. (min)	2.24-2.64	2.56
Freezing-Thawing % loss (max)		
Rock Fragments (max)	10-14	---
	(16-25 cycles)	
Rock Slab		10
		(12 cycles)
Wetting-Drying % loss		
Stone Slab		0
		(35 cycles)

3.4 ROCKFILL MATERIAL DESIGN AND CONSTRUCTION CONCERNS

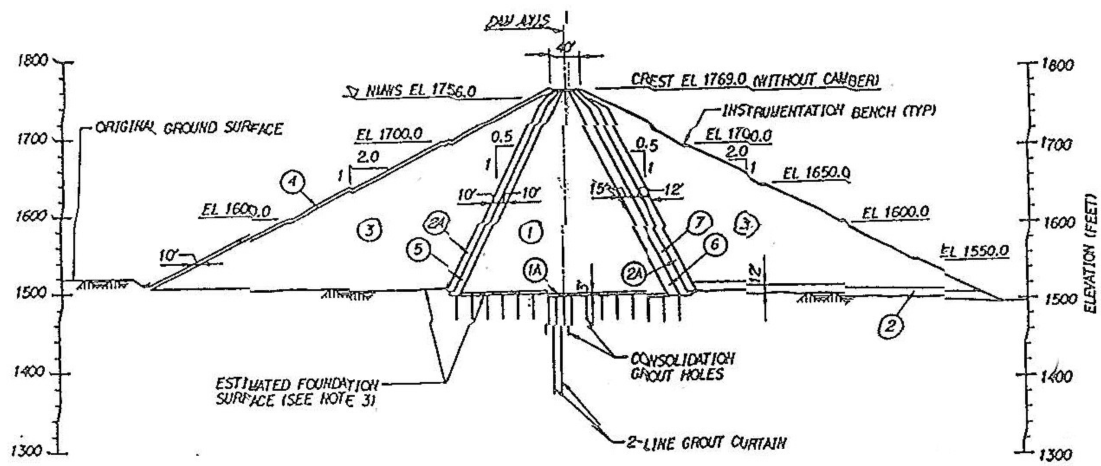
The economic design and construction of rockfill materials in dams depends on utilizing on-site materials without extensive processing and excessive excavation of unusable material. Many quarries for rockfill materials require considerable excavation of overburden and weathered rock before clean rockfill can be obtained. Recent design and construction of large dams has demonstrated that the weathered and weak rock can be used by placing materials where their engineering properties are compatible with design criteria. The selection of the zoning should accommodate the order that the materials will be produced. The overburden and weak rock (dirty rockfill) needs to be used early in the construction to minimize stockpiling for later use. The quarry for the West Dam of the Eastside Reservoir Project in southern California produced an estimated 45 million cubic yards of material with very little unusable material. Table 3.3 shows the types of material produced. Figure 3.1 shows a typical cross section of the dam on both an alluvial and rock foundation with specified zonations.

Table 3.3. Types of Material Produced, West Dam

Zone	Description	Materials	Source	General Material Description
1	Core	Silty Sand, Sandy Silt, Clayey Sand & Sandy Clay	BA1W1 and BA1W2	6” max., 20% to 80% passing #200 sieve
1A	Core at Cutoff Connection and Base of Core	Clayey Sand & Sandy Clay	BA1W1 and BA1W2	3” max., 20% to 80% passing #200 sieve, min. PI = 5
2	Blanket Drain	Quartzite	BA3	30” max., ≤10% passing #4 sieve
2A	Coarse Filter	Quartzite	BA2	6” max., ≤5% passing #4 sieve
3	Shell	Quartzite & Phyllite	BA2, BA3 & I/O Channel	30” max., ≤20% passing #4 sieve
3A	Stripping Rock Shell	Weathered Quartzite & Phyllite	BA2, BA3 & I/O Channel	9” max., , ≤35% passing #200 sieve
3B	Filter	Weathered Quartzite & Phyllite	BA2, BA3 & I/O Channel	3” max., 5% to 25% passing #200 sieve
4	Upstream Slope Protection	Quartzite & Phyllite	BA2, BA3 & I/O Channel	Approx. max diameter 3.4 ft., approx. min diameter 1.1 ft.
5	Upstream Filter	Gravelly Sand	BA2	1½” max. ≤3% passing #200 sieve
6	Downstream Filter	Sand	BA2	3/8” max. ≤2% passing #100 sieve
7	Drain	Gravel	BA2	2” max, ≤2% passing #8 sieve
8	Random Fill	Earth and/or Rock	Required Excavation, Rock Borrow Overburden	9” maximum



Typical Dam Section on Alluvial Foundation



Typical Dam Section on Rock Foundation

Figure 3.1. West Dam Cross-Sections

Test blasts and test fills are performed of potential rockfill materials from quarries for major embankment dam projects to determine the suitability and engineering properties of the excavated materials.

Materials obtained from alluvial deposits also require selection and consideration in zoning to minimize the production of unusable material. Rockfill materials (boulders, cobbles, gravel and fines) were obtained for several large dams in California from alluvial deposits reworked by bucket elevator dredges in the late 1800s and early 1900s. These dredges separate the sand from the original deposits and deposited the sand in dredge ponds in a slurry, from which the sand and fines settled to the bottom. The coarse gravels were deposited over the sand and fines. As a result, the potential borrow area for rockfill consisted of coarse gravel overlaying sandy materials. With selective excavation rockfill and transition materials were obtained. Table 3.4 shows the average type of material obtained for Oroville Dam in northern California. Figure 3.2 shows the cross-section of Oroville Dam.

Table 3.4. Average Type of Material, Oroville Dam

Zone	Maximum Particle Size	% Passing No. 4 Sieve	% Passing No. 200 Sieve	Comment
2	6 cm	25	5	Transition Material
3	9.5 cm	14	5	Shell
5	9.5 cm	8	3	Drain

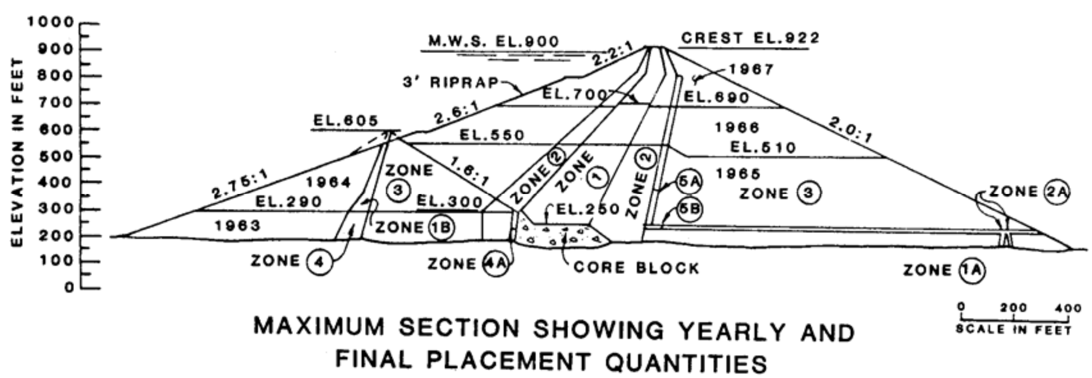


Figure 3.2. Oroville Dam Cross-Section.

Processing of rockfill materials by crushing, screening, and washing is required to produce materials with narrow specification limits on gradations for drain and filter zones. These operations are expensive and require special care in design and construction to use the materials as efficiently as possible.

Rockfill materials are placed in an embankment as individual cones, windrows or end dumps spaced to yield a layer of designed thickness after spreading. The maximum allowable lift thickness is determined by the maximum particle size. The lift is usually 1.2 to 1.5 times the maximum particle size. The maximum allowable particle size is determined by the gradation of the potential rockfill and the compaction required. For major projects, test embankments are constructed to confirm design assumptions for placing and compaction procedures.

During construction, approximate layer thickness and number of compaction passes required is routinely confirmed by inspection. Acceptance of rockfill is based on judgment. Grading and compacted unit weight tests are performed for records. Occasionally, if there are doubts and low compressibility is desired, lift thickness can be reduced or compaction effort increased.

Ramping within the embankment is acceptable and economical during placement of rockfill. Pinto (1985) and Cooke (1985) To improve construction procedures, accesses and ramps can be included in contract documents, or required as a submittal from the Contractor for approval from the Owners' engineer, as long as site conditions for accommodating selected hauling units are considered. When rockfill dams with impervious cores are built in areas with well-defined wet and rainy seasons, the rockfill may be placed well in advance of the core during the rainy period. Guavio Dam is an example of ramping inside rockfill shells to allow uninterrupted construction of the dam.

Once ramping is completed, standard procedure is to remove about 1.5 m of loose material along the slope before placing the next rockfill layer. The excavation is performed just before placing the new rockfill, to minimize loosening of already compacted material.

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CHAPTER 4 — GRANULAR FILTERS AND DRAINS

4.1 INTRODUCTION

The material for this chapter is taken from ICOLD Bulletin 95, *Use of Granular Filters and Drains in Embankment Dams* (1994).

The importance of filters and drains within the body of an earth or earth-rock dam must be uppermost in the mind of the embankment dam designer. Indeed, many incidents of failure or near failure can be attributed to the absence of filters and/or drains or to filter protection that was not appropriate to the application. The literature within the various ICOLD Congress proceedings and other sources provides ample case histories of incidents related to the lack of proper filter protection. Depending on the data cited, 30 to 50 percent of accidents to embankment dams have involved piping or inadequate drainage.

The safety of earth and earth-rock dams depends to a large degree on the proper design, construction, and maintenance of filter and filter/drain systems.

4.2 UNDERLYING PRINCIPLES

4.2.1 Basic Requirements of Filters and Drains in Dams

Two fundamental functions are required of filters and drains in earth, earth-rock, and rockfill dams:

1. **Retention function:** The filter must prevent migration of soil particles from adjacent foundation or fill materials. Thus, a fine filter must prevent migration of finer-grained impervious fill or foundation material; a coarse filter or drain must prevent any tendency for movement of the fine filter. This first requirement is often referred to as the piping or stability criterion. More recently, the term retention criterion has been used.

The classic Terzaghi criterion $D_{15}/d_{85} < 4$ addresses this requirement. In this expression the following symbols are used:

.

D_{15} = particle size in filter (protecting, or coarser material) for which 15 percent by weight of particles are smaller; and

d_{85} = particle size in base (protected, or finer material) for which 85 percent by weight of particles are smaller.

2. **Permeability function:** The filter must accept seepage flows from adjacent foundation or fill materials without the buildup of excess hydrostatic pressure. Thus, a fine filter must readily accept seepage flows from a finer-grained impervious fill or foundation material; a coarse filter or drain must readily accept flow from an adjacent fine filter. Permeability ratios between adjacent materials of at least 25 are often quoted.

To allow for variations of the base material and conditions of anisotropy as a result of compaction of the filter (which generally reduces vertical permeability) an average permeability ratio of over 100 may be needed. Special care must be taken if transverse cracking of the core (base) material is assumed to occur.

The classic Terzaghi criterion $D_{15}/d_{15} > 4$ addresses this requirement. If core cracking is a possibility, and in many instances it must be assumed, then high capacity drains with appropriate filters are required.

In practice, these two requirements have occasionally been in conflict. In the interest of preventing internal erosion, a filter is constructed with permeability close to the adjacent fine-grained fill or foundation material. In this instance, high pore pressure can develop and seepage that enters the filter system is not readily carried away. Seepage may be forced elsewhere.

Conversely, a filter, improperly designed or constructed, may be too coarse for the adjacent fine-grained material or the filter becomes segregated with coarse material at the interface. This can lead to excessive movement of the adjacent fine material into the filter. Specific instances of these or similar problems abound in the literature.

To achieve the above functions, the ideal filter or filter zone will:

- **Not segregate** during processing, handling, placing, spreading or compaction. The filter gradation must be sufficiently uniform such that, with appropriate care in the field, segregation is avoided in the placed material, especially at the interface between adjacent materials. The prevention of segregation is most important.
- **Not change in gradation** (degrade or break down) during processing, handling, placing, spreading and/or compaction; or degrade with time as might be caused by freeze-thaw or seepage flow. The filter must consist of hard, durable particles not susceptible to degradation as a result of slaking, weathering or other mechanisms. Breakdown of material causes changes to the permeability; this becomes critical when the filter is acting both as a filter and a drain. The gradation of the material is measured after placement and compaction. This test includes the effects of particle crushing caused by transportation, handling and compaction, and contamination by surface runoff, dust or other construction activity.
- **Not have apparent or real cohesion** or the ability to cement as a result of chemical, physical or biological action. The filter must remain cohesionless so that no tendency to crack exists even though cracking may have damaged an adjacent core zone.
- **Be internally stable**, that is, the coarser fraction of the filter with respect to its own finer fraction must meet the retention (piping) criterion. If the material is broadly graded, segregation in handling and placement is more likely and internal stability can become a serious problem.
- **Have sufficient discharge capacity** such that seepage entering the system is

conveyed safely and readily with little head loss. Thus, chimney and blanket filter/drain systems must be designed with ample discharge capacity. The design of chimney and blanket drains should consider the worst scenario; this might include a cracked core, hydraulic fracturing, and/or core segregation. To achieve both the requirements of retention for the range of gradation of the base and to provide adequate seepage discharge capacity, a single narrow zone can rarely suffice. Most often, a fine filter and a free draining zone combination are required. The required minimum permeability and thickness of filter and drain layers should be selected based on the use of Darcy's Law to calculate probable quantities of seepage that must be discharged.

- **Have the ability to control and seal a concentrated leak** through the core. The U.S. Soil Conservation Service "No Erosion Test" (Sherard and Dunnigan, 1985, 1989) may be used to determine this ability.

4.2.2 Flow Conditions Acting on Filters

The two flow conditions that typically act on filters are:

1. Flow perpendicular or approximately perpendicular to the interface:

    - At the downstream contact between the core and fine filter in an earth, earth-rock or rockfill dam
    - At the upstream contact between the core and fine filter in an earth, earth-rock or rockfill dam, locations subject to a fluctuating reservoir (flow from core to filter during reservoir drawdown)
    - At the contacts between the fine filter and coarse filter (drain) in downstream chimney, blanket and finger drains
    - At the contact between foundation soils and the bottom filter layer in a downstream blanket filter/drain or finger drain system
    - At the contact between earthfill and the top filter layer in a downstream blanket filter/drain or finger drain system
    - At the contacts between sand-gravel layers and silt-clay layers within alluvial foundations near the upstream and downstream toes of embankment dams, locations where seepage flows are perpendicular or nearly perpendicular to the slope of the layers

2. Flow parallel or approximately parallel to the interface:

    - At the contacts between bedding filters and base material, and between bedding filter and riprap or revetment on the upstream slopes of embankment dams
    - At the contact between gravel-cobble slope protection and base material on the downstream slopes of embankment dams
    - At the contacts between sand-gravel layers and silt-clay layers within alluvial foundations below embankment dams, locations where seepage flows are parallel or nearly parallel to the slope of the layers

- At the contacts between coarse filters and fine filters within high flow capacity filter/drain blankets on downstream foundations

4.3 FILTER RESEARCH

The references to this chapter list the work of many researchers in filter design, including Kenney, Lafleur, Vaughan, Brauns and their colleagues.

Sherard's great interest in cracking and piping in embankment dams is amply described in his writings (Sherard; 1973, 1979, 1985). This interest led to the research conducted in the Lincoln, Nebraska, soil mechanics laboratory of the U.S. Soil Conservation Service (USDA SCS — now Natural Resources Conservation Service) during the early 1980s. Their work has been widely reported (Sherard et al, 1984a, 1984b, 1985, 1989) and is now included in the design criteria for filters by the Natural Resources Conservation Service, the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers (USDA SCS, 1986; USBR, 1987a; USCOE, 1994). The basic conclusions from this research are presented in Tables 1 and 2.

In correspondence concerning the SCS criteria for filters, Talbot states (Talbot, Appendix B, Bulletin 95):

> *"Besides the criteria presented in Table 1, the note (Soil Mechanics Note No. 1, Guide for Determining the Gradation of Sand and Gravel Filters) contains 10 steps explaining how to design a filter. Step 10 indicates that for some soils, the filter gradation limits obtained using the criteria can be rather broad so as to allow the use of skip-graded or gap-graded filters, and that a narrow band should be specified within the broad limits to prevent this.*
>
> *"After using our guide for six years, SCS has decided to make some refinements and insert it into the SCS National Engineering Handbook. We believe it is important to use relatively uniform granular materials and to ensure gap or skip graded materials are not used. The final draft of our revised guide is in the last stages of preparation. It includes requirements that the coarse and fine sides of the specified filter band have a coefficient of uniformity (D_{60}/D_{10}) of six or less. Also, the filter band must be narrow such that the ratio of maximum particle size to minimum particle size is five or less at all percent passing values of 60 or less. The narrow band defining uniformly graded material may be located anywhere within the broad limits defined by the criteria."*

Table 4.1. Criteria for Filters (USDA SCS, 1986; USBR, 1987a; USCOE, 1994)

Base Soil Category	Base Soil Description, and Percent Finer than No. 200 (0.075 mm) sieve 1/	Filter Criteria 2/
1	Fine silts and clays; more than 85% finer	$D_{15} \leq 9 \text{ x } d_{85}$ 3/
2	Sands, silts, clays, and silty and clayey sands; 40 to 85% finer	$D_{15} \leq 0.7$ mm
3	Silty and clayey sands and gravels; 15 to 39% finer	$D_{15} \leq \frac{40-A}{40-15}(4xd_{85}-0.7mm)+0.7mm$ notes4,5
4	Sands and gravels; less than 15% finer	$D_{15} \leq 4 \text{ x } d_{85}$ 6/

1/ Category designation for soil containing particles larger than the #4 sieve (4.75 mm) is determined from a gradation curve of the base soil which has been adjusted to 100% passing the No. 4 (4.75 mm) sieve.

2/ Filters are to have a maximum particle size of 75mm (3 inches) and a maximum of 5% passing the No. 200 (0.075 mm) sieve with the plasticity index (PI) of the fines equal to zero. Note that the criteria relating the D_{90} to the D_{10} shown on Table 4.2 must be used to design the filter gradation ranges. These criteria force the designer to use uniform filter gradations that help to prevent segregation during placement. PI is determined on the material passing the No. 40 (0.425 mm) sieve in accordance with ASTM-D-4318. To ensure sufficient permeability, filters are to have a D_{15} size equal to or greater than 4 x d_{15} but no smaller than 0.1 mm.

3/ When 9 x d_{85} is less than 0.2 mm, use 0.2 mm.

4/ A = percent of base material passing the No. 200 (0.075 mm) sieve after any regrading.

5/ When 4 x d_{85} is less than 0.7 mm, use 0.7 mm.

6/ In category 4, the d_{85} may be determined from the original gradation curve of the base soil without adjustments for particles larger than 4.75 mm.

Table 4.2. D_{10f} and D_{90f} Limits to Prevent Segregation
(USDA SCS, 1986; USBR, 1987)

Minimum D_{10} mm	Maximum D_{90} mm
<0.5	20
0.5 - 1.0	25
1.0 - 2.0	30
2.0 - 5.0	40
5.0 - 10	50
10 - 50	60

The above is now included in the design criteria adopted by the SCS, now called the Natural Resources Conservation Service (McCook, Talbot, 1995). Other conclusions of the research included:

1. No relationship or correlation was found between D_{50} and d_{50} or between D_{15} and d_{15} for filter performance. Previous filter criteria employing D_{50}/d_{50} or D_{15}/d_{15} relationships should be abandoned. Requirements of D_{15}/d_{15} equal to four or more may be used to assure adequate permeability of the filter, but are not needed to define filter properties.
2. For typical coarse glacial moraines, graded from cobbles to fines, and other similarly graded impervious soils, the USDA SCS research demonstrated that a sand or gravelly sand with D_{15} # 0.7mm is needed for a conservative downstream filter.
3. The No Erosion Test "was found to be the best test for routine laboratory evaluation of filters for specific projects. It is applicable for tests on coarse impervious soils as well as fine clays and silts." (Sherard, et al, 1985).
4. "Both recent extensive laboratory research and evaluation of experience with dam behavior support the conclusion that adequate filters will reliably seal and control concentrated leaks through earth cores of embankment dams." (Sherard, et al, 1985).

4.4 DESIGN REQUIREMENTS OF FILTERS[3]

4.4.1 Base Soil Analysis

Before a filter can be designed, the grain-size distribution curve of the base soil must be analyzed. This analysis must determine whether or not the material is broadly graded and potentially internally unstable. Materials that may be internally stable in the controlled environment of the laboratory may be internally unstable in the field if segregation occurs such that pockets or lenses of differing gradations exist within the fill. Substantial differences in gradation can occur during dumping and spreading of broadly-graded materials or at the interface between adjacent zones, or if compaction is poor, such as adjacent to instrument locations or in trenches.

A means to determine the internal stability of a base soil was suggested by Sherard (1979) and de Mello (1975). Lowe states (Jansen, p 270, 1988):

> *"It has been found that the Terzaghi criteria can be used to check the self-filtering ability of broadly-graded and skip-graded materials. The material can be checked by separating the grain-size curve into two parts at any arbitrary point of separation, as indicated in Fig. 1. For a self-filtering material the D_{15} size of the coarser fraction should be no more than 5 times the d_{85} of the finer fraction. As is evident from the figure, whenever the slope of the grain-size curve is flatter than 15% per a 5 times change in grain-size, the material is not self-filtering. When such is the case, then the D_{15} of the filter should be based upon the d_{85} of the finer fraction rather than on the d_{85} of the total material."*

Suggested adjustments to the base material are summarized below:

Organization/Individual	Base Material Adjustment
J. L. Sherard, USDA SCS, USBR	Fraction finer than 4.75mm
J. Lowe (Jansen, 1988)	Analysis of gradation curve
J. Lafleur (Lafleur, et al, 1989)	Analysis of gradation curve

For the Cat Arm hydroelectric development in northern Newfoundland, the analysis of the gradation distribution of the broadly-graded moraine core material led to a filter design which utilized the fraction finer than the #100 sieve, 0.15mm (Humphries and Connors, 1989). This resulted in a fine filter with a maximum D_{15} = 0.5mm.

[3] Note that gradation of the filter or of the base material is the in-place gradation, which includes the effects of particle crushing as a result of transportation, handling, compaction, and the stresses imposed by overlying material in the dam; and contamination caused by surface runoff, dust, and construction activity.

4.4.2 Retention Criterion

All practicing engineers and researchers place great emphasis on the selection of the retention criterion, ie., the "piping" or "stability" criterion. The several proposals that appear most frequently in the literature include:

1. For the fine filter adjacent to impervious fill or fine-grained soil foundation, use a sand or sand-gravel filter with top size of 12.7 to 19.0mm (1/2 to 3/4 inches) with 55-80 percent passing the #4 sieve. The material should be well graded from the maximum particle size to the fine sand sizes with no more than 5 percent passing the #200 sieve. An alternative gradation is the ASTM C33 gradation for fine concrete aggregate, as shown in Table 3. This results in D_{15} sizes from about 0.2 to 0.5mm.

Table 4.3. Gradation — Fine Concrete Aggregate

ASTM Sieve Size mm		% Passing by Weight
9.50	3/8"	100
4.75	#4	95-100
2.36	#8	80-100
1.18	#16	50-85
0.60	#30	25-60
0.30	#50	10-30
0.15	#100	2-10

2. Use the classic Terzaghi relationship,

$$D_{15}/d_{85} < 4$$

After appropriate adjustment to the shape of the gradation curve of the base. The choice of the d_{85} size is based on analysis of the base gradation curve.

3. Use the procedure developed by Sherard and his co-workers at the U.S. Soil Conservation Service (Sherard, et al, 1989). In this method the base material is adjusted as required and categorized. The filter is then designed according to the rules stated in the guideline (USDA SCS, 1986[2]).

For major projects or for projects with questionable materials, it is prudent to perform a series of laboratory filter tests to substantiate the selection of the most appropriate filter. Where dispersive soils are present, laboratory filter tests should be performed. Use of the

[2] The NRCS has modified their criteria to avoid gap grading of filters and to achieve appropriate uniformity of the gradation. The most current criteria is contained in Chapter 26, Gradation Design of Sand and Gravel Filters, Part 633, National Engineering Handbook, USDA, Soil Conservation Service (now the Natural Resources Conservation Service).

criteria developed by the USDA SCS, 1986, i.e., a fine sand filter with D_{15} between 0.1 and about 0.3 mm will, in most cases, provide an adequate filter for dispersive soils.

4.4.3 Permeability Criterion

The filter must accept seepage from the adjacent embankment or foundation without the build-up of excess pore pressure. The guideline most often quoted and used is the Terzaghi relationship:

$$D_{15}/d_{15} > 4, \text{ or } > 3 \text{ to } 5 \text{ (Corps of Engineers, 1994)}$$

This ensures a ratio of permeability of about 20 times between the filter and the adjacent base material, since permeability varies approximately with the square of the D_{15}. In addition to the above criterion, the U.S. Soil Conservation Service (USDA SCS, 1986) adds the requirement that the D_{15} must be no finer that 0.1mm.

As indicated by Cedergren, even minute quantities of silt or clay can greatly diminish the permeability of sands (Cedergren, 1977). A limit on the percentage of minus #200 material, determined on samples taken after compaction, should be clearly stated in the specifications. Percentages from two to about seven percent by weight of non-plastic fines are currently allowed depending on the characteristics of the material source. A five- percent maximum limit is most often used.

For important structures, laboratory permeability tests should be performed on all materials to ensure that the specified gradations are acceptable.

4.4.4 Discharge Capacity

It is imperative that filter and drainage systems within embankment dams safely conduct all seepage water to the downstream toe or to an adjacent more pervious zone without the buildup of excess pressure. Design of drainage systems should consider a worst-case scenario that includes core cracking, hydraulic fracture, and/or core segregation. With ample discharge capacity, the line of seepage will not rise above the horizontal downstream drain connected to the chimney drain. In his response to the questionnaire for Bulletin 95, Cedergren states:

> *"I believe this is one of the most important topics for the bulletin, because many dams throughout the world have been built in the past 20-30 years (and even now) with drains incapable of removing seepage without large buildup of pressure. Most dam designers seem to believe that if a filter or drain is designed so that the D_{15} of the filter (or drain) is at least 4 or 5 times the d_{15} of a protected soil it will have adequate discharge capacity.*
>
> *"...While there are a number of ways of analyzing flow in filters and drains, one of the simplest to use (after potential flow rates have been estimated) is Darcy's law in the form:*

$$Q/i = kA$$

"In this equation, Q is the estimated rate of flow which must be handled by the filter or drain (per unit length of structure), i is the allowable (available) hydraulic gradient in the filter or drain, k is the required coefficient of permeability of the filter or drain having an area, A, normal to the direction of flow in the filter or drain. Any practical combination of k and A that ensures the required discharge capacity (with an adequate factor of safety) can be used. Generally, relatively thin layers of highly permeable materials are more economical than thicker layers of lower permeability material (in the conducting elements of drains)."

An ample safety factor should be used to consider:

- The worst-cracking scenario for flow through the core
- The approximations in the estimate of the in-situ permeability of the filter and drain system
- The variation of permeability with the amount of fines after compaction
- The anisotropic permeability characteristics of the system, vertical permeability for the chimney drain, horizontal permeability for blanket and finger drains

4.4.5 Segregation

Filter or drain material must not segregate during construction. The processing, handling, stockpiling, re-excavation, dumping, spreading, or compaction of the filter material must be carried out to minimize segregation. Construction methods must be specified, planned, executed, and confirmed by continuous inspection and field testing to assure that segregation does not compromise filter or drain performance.

The filter gradation must be sufficiently uniform to preclude segregation. The current design standards of the U.S. Soil Conservation Service (USDA SCS, 1986) and the U.S. Bureau of Reclamation (USBR, 1987) include the criteria presented in Table 4.2 which are to be used with the guidelines shown on Table 4.1. The coarser filters must be more uniformly graded to avoid segregation during construction. The recommended Uniformity Coefficient varies from about 6 for filters consisting of sand with some gravel sized particles to about 3 for coarse filters or drains with top sizes on the order of 75mm (3 inches). When thin filters are used, i.e., on the order of 1m or less, close control of segregation is mandatory.

The use of the U.S. Soil Conservation Service procedure can lead to the design of a filter with a broad range of particle sizes that could result in allowing the use of gap-graded materials. These materials have a grain size distribution curve with sharp breaks or other undesirable characteristics and may be susceptible to segregation during placement. The Natural Resources Conservation Service has completed a revision to their criteria for the design of sand and gravel filters, (McCook, Talbot, 1995). The following criteria have been added as a final step:

1. Adjust the limits so that the coarse and fine sides of the filter band have a Uniformity Coefficient of 6 or less.
2. Adjust the limits so that the width of the filter band results in a ratio of maximum diameters to minimum diameters of 5 or less, at any given percent passing of 60 or less.

4.4.6 Self-Healing by Collapse

Filter and drain material must be cohesionless and be capable of collapse and self-healing should cracking occur even though an adjacent core zone may have been damaged by cracking. Vaughan (1982) suggests the use of the "sand castle" test for cohesion:

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

Filter or drain material should not gain cohesion or "cement" with time. Certain materials may gain cohesion with time and access to moisture. As Vaughan suggests, the "sand castle" test can be used to evaluate the tendency of a filter material to gain cohesion with time. The U.S. Soil Conservation Service criterion (USDA SCS, 1986) of $D_{15} > 0.1$mm also ensures a cohesionless filter, unless clayey fines or a carbonate cement is present.

4.4.7 Ability to Control a Crack in the Core Material

Since there is no way to assure *a priori* that the core will not crack, the ability of the filter material to control a concentrated leak through the core material should be determined by conducting the USDA SCS No Erosion Test (Sherard and Dunnigan, 1985, 1989) using the D_{15} size of the proposed filter material as a quantitative measure of its ability to control and seal a concentrated leak through the core.

4.4.8 Quality

In general, good quality filter or drain material will consist of hard, durable particles which will not degrade as a result of chemical, physical or biological action. Materials must be avoided that will:

- Gain cohesion, cement or clog with time as a result of chemical or biological attack
- Change gradation as a result of the manufacturing, placement, and compaction

process

- Change gradation with time as a result of the freeze-thaw process
- Change gradation under high compressive and shear stress as exists at the base of high embankment dams

Crushed or natural material or mixtures of both can be used for filters. Test procedures that are normally used to evaluate the quality and soundness of potential sources for concrete aggregate should be used to test prospective filter material. Laboratory tests for gradation must include washing to determine the percentage of material passing the #200 sieve. Field tests, with accompanying gradation tests, are used to assess the breakdown caused by compaction and handling.

Limestone rock filters in an acid environment will degrade with time. Plugging of outlets with an iron bacterial slime or other chemical or bacterial attack can occur. Deposits coming out of solution can plug and cement filters, thus, destroying the function of the filter/drain system.

4.4.9 Critical Filter to Protect the Core

Sherard and Dunnigan (1989) describe laboratory research directed at studying "critical" downstream filters that have been or might be exposed to concentrated leaks developing in the protected impervious embankment material. They state:

> *"A wide range of different fine silts and clays and clayey and silty sands of different geologic origins were tested [using the "No Erosion Test" described earlier in this bulletin]. The results of the investigation confirm conclusively that sand filters containing appropriate quantities of fine sand will reliably control and seal concentrated leaks through the impervious sections of embankment dams. The investigations show that for most fine silts and clays, a downstream sand filter with D_{15} < or equal to 0.5mm is conservative and that broadly-graded soils, such as those from glacial moraines, need a relatively fine filter."*

Perhaps Peck (1990) presents the best summation:

> *"The writer agrees with Sherard that the available data on performance of dams suggest that the filter adjacent to the core serves a purpose even more vital than has generally been assumed.* ***Undoubtedly, as has long been recognized, the downstream filter limits the amount of material that can be lost from the core by erosion and thereby protects the integrity of the core. In addition, however, it serves as a substitute core where, for any reason, defects in the core have permitted concentrated seepage.*** *Hence, seepage through the dam as a whole may not increase perceptibly even when defects develop in the core. The impregnated filter takes over the function of the core.*
>
> *The writer would not go so far as Sherard in concluding that this beneficial action of the filter would justify reducing efforts to ensure the integrity of the core. Arthur*

> *Casagrande's principle of defense in depth, so wholeheartedly adopted by Harry Seed, should remain a guiding principle for all designers of dams.* ***The lesson to be learned is not that cores may now be considered less vital with respect to safety, but that filter protection deserves greater care and attention.*** *In any event, consideration of the importance of the filter skin, the impregnated upstream portion of the filter zone, should lead to improved understanding and design of water-retaining structures."*

4.5 CONSTRUCTION PRACTICE

4.5.1 Width of Filters

Laboratory and field tests indicate that the filtering ability of an appropriately graded filter occurs within a few centimeters of the interface with the protected material. Thin filters, on the order of 1m wide, are used if processed material is available. Use of such thin filters assumes a material with closely controlled gradation and quality at the manufacturing plant and careful handling and placement techniques that avoids segregation. "Narrow filters are difficult to place except in a vertical trench or with spreader box and are not accepted in a most critical earth dam (effect of 'Christmas tree' boundary) and generally the minimum practical recommended width varies between 2 and 3 m or even more in special cases (earthquake areas)" (private communication, Georges Post, 1993).

Use of wide transition zones with broad material gradation such as unprocessed material from natural deposits, should be avoided. These less controlled materials will suffer segregation during placement and are likely to vary in gradation and quality; such variations are especially critical at the interface with adjacent material.

Where embankment dams are subject to strong earthquake shaking, displacements or permanent deformations may occur. The design of such structures should be carefully evaluated with respect to width and thickness of filters and drains. The design should assure that if a filter or drain zone is displaced, sufficient width and thickness of that zone remains so that its hydraulic capacity is not critically reduced.

4.5.2 Compaction Requirements

The compaction of filters should be adequate to produce sufficient density to preclude liquefaction, to limit consolidation, and to provide adequate strength. Excessive compaction can cause particle breakdown, reduce permeability, and in some cases increase the percent of fines to an amount greater than the specified limits. The USBR and the New South Wales Public Works Department in Australia suggest a minimum 70 percent relative density. Density tests of in-place material should be made in the second or third lift below the current surface because the vibration of the material leaves the upper part of the first lift in a relatively loose state. Filters should be compacted to the degree necessary to have their compressibility approximately the same as that of the adjacent materials. Filter material should be moist at the time of compaction to avoid "bulking" of the filter material.

4.5.3 Segregation Problems

Segregation during placement is a common problem that often results in overly coarse filter and drain material in contact with an adjacent finer material. Incompatibility at the interface between materials is the result.

Segregation of filter material is avoided by:

- Using narrowly-graded uniform filters
- Limiting the maximum particle size of the filter

If the USDA SCS criterion for control of the D90 with respect to the D10 is used in selecting the gradation, Table 4.2, segregation of the filter should not normally be a problem.

4.5.4 Filter Contamination

Contamination of filters during construction is a common problem. Crossing the filters and drains with construction equipment can contaminate or move the filter or drain material causing serious damage and disruption of the filter width. Costly repair can result. It is common practice to restrict the crossing of filter zones to specific locations, thus reducing the exposure of filters to damage. Where crossings occur, the filter/drain system should be protected; geotextiles, geomembranes, and/or a protective fill, 0.3m thick, are commonly used. Crossings should be moved horizontally from lift to lift, especially adjacent to abutments, to avoid a vertical zone of material that has a higher probability of contamination and excessive compaction.

Contamination also occurs during periods of rainfall when muddy runoff carries fines to the filter zones. This can occur from erosion of a partially constructed fill surface or from erosion of the abutment areas. If the fine filter is designed properly, only a thin skin of fines will cover the surface of the filter; after removal of the skin, the intact uncontaminated filter is exposed. Little damage has occurred. If the fine filter is too coarse, eroded fines will enter the filter and contamination will occur. Contamination of the coarse filter or of drain material with eroded fines can and often does occur. In this case, fines will penetrate the material because of the coarseness of the material. Keeping the elevation of the filters higher than the adjacent fill can minimize such problems. Sloping the embankment surface away from the filters will force runoff toward the outer slopes of the embankment and away from the filters.

Contamination with dust can occur when dumping adjacent rockfill without water sluicing or at an excessively dusty location such as adjacent to a haul road with little dust control.

Protection with a geotextile or geomembrane is a technique that is often used during prolonged periods of inactivity, during persistent rainy periods, or when the filter is otherwise vulnerable to contamination.

4.5.5 Change in Properties during Construction

The construction operations of placing, spreading, and compaction can influence the quality of the constructed filter. Improper placing and spreading can cause concentrations of oversize particles too coarse to protect the base material. Marginally durable filter materials may break down during compaction resulting in a filter with excessive fines, reduced permeability, and the ability to sustain a crack.

4.5.6 Quality Control Tests

Control tests of the gradation and quality of the filters should be performed **after compaction**. Acceptable sources of materials with respect to quality should be approved prior to construction.

4.5.7 Unclear Specifications

Pritchett (1985) suggests that the specifications should cover the following in addition to the usual requirements for compaction, lift thickness, gradation, and material quality:

- Placement and spreading techniques, such as keeping the filter zones at a level higher than the adjacent fills
- Moisture conditioning and spreading requirements to minimize segregation and inter-zone intrusion such as the use of spreader boxes
- Requirements for deliberate compaction of zone boundaries

Other requirements might include:

- Locations and measures to protect crossings of filter and drain systems
- Required protection during prolonged shut down of operations

4.5.8 Inadequate Construction Surveillance/Inspection

Pritchett (1985) emphasizes the necessity to properly specify, inspect, and assure that the actual construction process translates the design intent into satisfactory “as-built” conditions.

Ingles (in response to the questionnaire, Bulletin 95) states:

> *"The safety factors in design seem to be quite adequate; those in quality control are certainly not. For earth dams, where the slightest lapse in quality can be fatal, I have long recommended that the cheapest form of quality assurance is a professional engineer on the site, on the job, every hour of the working day."*

Commonly, the owner/designer organizes a field team to perform the tasks of quality control and quality assurance to enforce the specifications and to provide surveillance during construction. The field staff must maintain close contact with the designers for decisions related to the design intent. In addition, the design team makes frequent trips to the site to provide additional input as needed and for routine inspection. The combined effort of these activities is to increase the success rate of translating the design intent to the actual constructed product.

4.5.9 Resultant Unsatisfactory Structure

Inadequate specifications and inadequate field surveillance can leave many critical decisions improperly made. The result is that the intended quality and function of the filters are easily compromised. Filters that are constructed under inadequate specifications and surveillance will not likely perform as intended and the long-term safety of the embankment dam containing such filters may be in doubt.

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CHAPTER 5 — ASPHALT CONCRETE (AC) AS THE WATER BARRIER IN EMBANKMENT DAMS

5.1 INTRODUCTION

Asphalt has long been used in the construction of dams: to grout foundations when running groundwater washes away Portland cement particulate grout; as a coating for conduits penetrating the dam, to control seepage; as a protective coating on exposed foundations that are subject to air or water slaking; and as the water barrier element of an embankment dam. This chapter will be devoted to the latter function, referring to Asphalt Concrete Cores and Asphalt Concrete Facings for embankment dams.

5.2 WATER BARRIER

The water barrier in an embankment dam is made up of three components: (1) the dam foundation; (2) the contact surface between the dam and its foundation; and (3) that part of the dam above its foundation that prevents or controls the seepage of water through the structure. This Chapter relates to the latter component, and will refer to it, alone, as the "water barrier."

An earthen core of low permeability is the common water barrier provided for a rockfill or zoned embankment dam. However, at many dam sites, particularly in high altitudes, impervious materials are very scarce, and those that are available are saturated. These facts can be complicated by a very short construction season. In these cases, an impervious earth core becomes impractical and an alternative solution is to design an asphalt concrete water barrier. This asphalt concrete diaphragm or membrane can be a vertical, or near vertical element situated in the interior of the dam, known as an asphalt concrete (AC) core; or an asphalt concrete facing constructed on the upstream face of the dam. An AC facing and an AC core differ in location, design concept and response to load. They each create a water barrier, but the facing additionally provides wave protection for the upstream slope. When an interior diaphragm is used for the water barrier, wave protection for the upstream slope must still be provided by rock-fill, rip-rap, soil-cement, concrete revetment, AC revetment, or other appropriate means.

5.2.1 The AC Core

AC cores, first developed in 1948, were sometimes constructed on a slope (upstream toward downstream, as the dam rises), but now are usually vertical in cross-section, and follow or parallel the axis of the dam, in plan. For high dams, the upper segment of the core may be sloped to maintain positive stresses on the core during reservoir operations. The modern AC core is placed right along with the rising embankment, keeping the fill crowned, with the core at the high spot so as not to be damaged by flooding during rainfall.

The modern cross-section, shown in Figure 5.1, is uncomplicated. The thickness of the AC diaphragm is often constant, and is 0.5m minimum, but not less than about 1 percent of the height of the dam. For high dams, the thickness can be reduced from the base to the crest, in steps. There are transition zones both upstream and downstream. The upstream transition, with a width of 1.5m to 3m, is grain-size compatible with the upstream shell, and has non-plastic fines that serve as a crack stopper for the AC core. The downstream transition, with a width usually ranging from 1.2m to 2m, functions as a chimney drain. In this arrangement the three materials are placed simultaneously in a tri-compartmented spreader box that is either self propelled or hauled along line. The three zones are compacted together and compaction is by vibrating roller or plate. The compactor for the AC is used only on the AC.

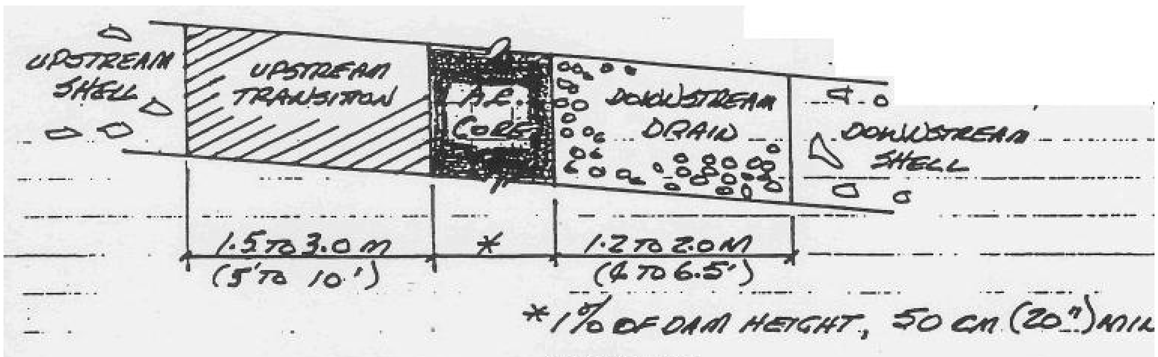


Figure 5.1. Cross-Section through a Modern AC Core

Great care must be exercised at the contact of the AC diaphragm and the dam foundation. There is either a plinth (with or without a grouting or instrumentation gallery) or a concrete sill at this interface. Establishing a seal between the plinth or sill and the foundation, and between the AC core and the plinth or sill, is essential.

ICOLD Bulletin 84 lists 62 dams with AC Cores (none in the U.S.). As compared to an asphalt concrete facing, the asphalt concrete core offers the following advantages:

1. It is simpler in detail. Only one asphalt concrete mix is involved.
2. It is simpler to construct. No fine grading of the subgrade is required. The filter, impervious membrane and drain are constructed in parallel, in a single placement and compaction operation; as opposed to the elements of a facing system, which are constructed in series.
3. It is more economical to construct.
4. It is permanently protected from aging, and therefore is virtually maintenance-free.
5. It cannot be damaged by impact.
6. It can be constructed in any weather suitable for construction of the embankment.
7. It experiences very little deformation when the water load is applied.
8. If damaged for any reason (e.g., abrupt differential settlement of the embankment; or earthquake), it is self healing.
9. It is constructed at the same rate as the rest of the embankment, does not delay construction, and is completed when the embankment is completed. In some

projects this can be very cost effective, compared to the facing, which construction usually is not started until after the dam is topped out.

The disadvantages of the core system, compared to the facing system are:

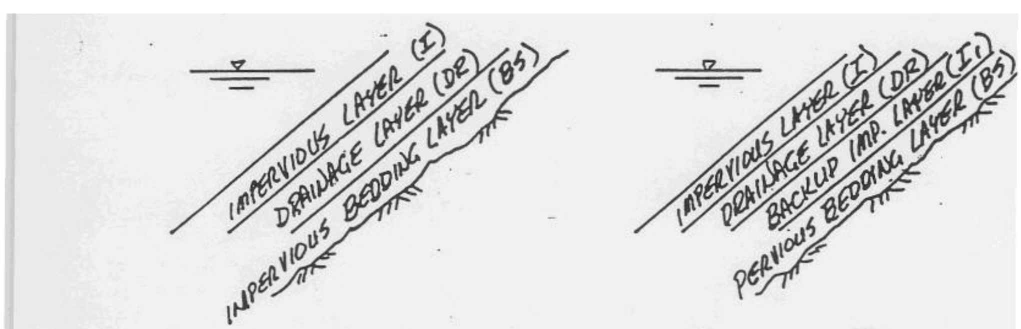
1. Once constructed, the core is difficult to access for inspection or repair.
2. Being in the interior of the dam, the core is incapable of doubling as protection against wave action on the upstream face of the dam and, so, for an earth-fill dam, rip-rap or other type revetment will be required.

In the case of an earth embankment, the fact that a revetment will be required on the upstream face has a significant cost impact. With rockfills, however, this is not a big issue, because the rip-rap is automatically derived from the oversize product of the quarry operation.

5.2.2 The AC Facing

The asphalt concrete facing, either for a dam or reservoir, is a composite diaphragm constructed as the water barrier on the upstream face.

The original designs were facings for pervious rockfills, and consisted of a single layer of asphalt concrete, made up of several lifts, placed and compacted on the upstream face. This design is no longer recommended, and current practice is that the composite system consists of several layers of asphalt concrete, each with a different mix and a different function. Although there are many variations of the design, the basic philosophy of the modern facing is that there be an outer **impervious layer**; underlain by a **drainage layer**; founded on a **bedding layer**. Ancillary coats include the **prime coat** on the face of the embankment, a **tack coat** between layers, and the **protective asphalt mastic seal coat** at the outer surface. The prime and tack coats are optional, depending on conditions and the discretion of the Engineer. The two most common systems in vogue are shown in Figure 5.2 as Types A and B. Type B is essentially the same as Type A, except that the bedding layer has two components, a **pervious base or leveling course** and a **backup impervious layer**. Both systems ensure that seepage through the outer impervious layer will remain in the drainage layer, where it can be monitored. The Type B should be used where there is any ground water or hydrostatic pressure to be relieved from under the facing or lining.



Type A Type B

Figure 5.2. Composite Diaphragm as an Asphalt Concrete Facing

In ICOLD Bulletin 114 (1999) on *Bitumimous Concrete Facings for Earth and Rockfill Dams*, there are 285 dams and reservoirs with this type water barrier listed, world-wide. Of these, 36 are in the U.S., including the very first, Central Dam, constructed in 1910. Some typical statistics are:

Slope of the embankment — generally in the 1.5:1 to 2:1 range. Stability of the hot asphalt concrete mix on the upstream slope of the embankment, at time of placing, is an important consideration.

Thickness of the composite system — there doesn't seem to be much agreement on this issue. Some follow the "1 percent of the height of the dam" rule-of-thumb, but there is little consistency between designs.

Impervious layer — constant thickness, varying from 5 to 10 cm.; 6 to 8 percent asphalt; densely graded aggregate; high compaction to about 3 percent air voids content; and impermeable.

Drainage course — constant thickness, varying from 8 to 20 cm; 2 to 4 percent asphalt; open graded aggregate; light compaction.

Base course — constant thickness, 7.5 to 25 cm (or more). Design depends on whether Type A or Type B sandwich is intended.

It is worthy to note that the recommended modern practice is to place and compact each layer to its full thickness, rather than placing in lifts. The resulting compaction is better, because the thicker lift holds the heat longer allowing more time for the roller to work. Also, the number of joints (which are often a source of trouble) is reduced.

Plinth and Boundary Conditions

The asphalt concrete facing is a relatively flexible structural system that, in the field, has no difficulty in deforming along with the dam. At the boundaries, however, the dam foundations and the plinth anchored into them, are rigid and offer the classic stress situation that can be expected whenever there is a connection of a flexible element to a rigid one. A similar situation often occurs at the crest where there is a parapet wall, although that location is seldom critical.

5.3 MATERIALS

The principal materials used in an asphalt concrete water barrier for an embankment dam are **asphalt**, **aggregate** and **filler**. Their combination is an **asphalt concrete mix**.

5.3.1 Asphalt

Asphalts used in highway paving construction are those that normally are used for dam facings or cores, and reservoir linings. The principal difference between a highway paving mix and a “hydraulic” mix used in a dam facing, reservoir lining or dam core, is in the percentage of asphalt in the mix. Typically highway mixes feature asphalt percentages in the order of 3.5 to 4.5 percent, whereas in hydraulic works the percentages are in the order of 6.0 to 8.0 percent.

In the U.S. asphalts are graded by four different systems: Penetration grades; Original Viscosity grades; “After hot mixing” Viscosity grades; and Performance grades. Both penetration and viscosity are a measure of the consistency of the asphalt. Penetration measures consistency at low temperatures, while viscosity measures it at higher temperatures. Performance grades refer to the stiffness of the asphalt at its highest service temperature, and its ductility at its lowest service temperature. The most common current specification for asphalt used in hydraulic structures is based on the Penetration grade. Each grading specification has its unique test procedures and requirements. However, any asphalt can be tested for compliance with the requirements for any grading system.

Penetration grading groups asphalts according to their performance in the Standard Penetration Test (ASTM D5 and D946). This grading method, in approximately its present form, dates back to 1910. The test is made on a sample heated to 25^{o}C (77^{o}F), and measures the vertical penetration of a calibrated needle, weighted by a 100g ballast, for a period of 5 seconds. The penetration of the needle into the asphalt is measured in units of 0.1 mm and is, by definition, the penetration of the asphalt. There are five penetration grades of asphalt on the market: **40-50, 60-70, 85-100, 120-150 and 200-300.** The first three are used in the construction of dam water barriers and reservoir linings, with the 85-100 grade being used more frequently. Typical asphalt contents for impervious elements are 6 percent to 8 percent asphalt, by weight of the total mix; and for the pervious elements are 2 percent to 4 percent.

Typically, an asphalt (a.k.a. bitumen, asphalt cement or asphalt binder)used in hydraulic structures should conform to the ASTM or AASHTO specifications for its grade. The asphalt selected for a project should reflect the site specific climate, such that the impervious layer of the mix will be stiff enough to be stable on the slope at the highest operating temperatures, and ductile enough to preclude thermal cracking at the lowest operating temperatures. That selection has traditionally been made on the basis of Penetration Grading, but the modern Performance Grading (AASHTO Designation MP1)shows great promise in the field of dam engineering because it offers 37 grades of asphalt that provide high temperature stiffnesses required under temperatures ranging from 46^{o}C to 82^{o}C, and low temperature ductilities required under temperatures ranging from -10^{o}C to -46^{o}C.

5.3.2 Aggregate

The term aggregate, as used herein, includes the *coarse aggregate*, having a grain size ranging from 4.76mm (#4 sieve) up to the maximum size, and the *sand*, ranging in grain size from 4.76mm down to 0.074mm (#200 sieve). Aggregates used in the mixes that make up the component layers of an asphalt concrete facing or the lifts of an asphalt concrete core are normally crushed stone, suitable for use in portland cement concrete.

The reports indicate that limestone is the rock most commonly used, but granite, quartzite, porphyry, diorite, gneiss and basalt have all been used successfully. The sound rock of the region is the most likely candidate, providing that it meets the requirements of ASTM C-33 or BS 882.

The important characteristics of the aggregate are its durability, chemical inertness, low porosity, particle shape and gradation. Durability and low porosity seem to go together. Regular, angular shapes (as opposed to slivers or plates) promote a stable pack. Workability may be enhanced by addition of natural aggregates, i.e., not crushed. The gradation depends on the function of the element. The impervious elements of a water barrier system require a well graded material, while the drainage layers require an open gradation. Good rules for the shape of the grain size distribution curves have been developed. Fuller's Curve or the U.S. Bureau of Public Roads "0.45 Power Curve" will yield the densest pack. The pervious mix for the drainage layer should be open graded coarse aggregate with little to no sand, and no filler.

5.3.3 Filler

By definition, *filler* consists of fines with a d_{max} of 0.074mm or, in other words, are 100 percent passing a #200 sieve. It is used to promote the workability and compactability of the mix. Workability is important, because harsh mixes, low in fines content, tend to "tear" when being placed. In addition to improving workability, proper quantities of filler also decrease the volume of voids, making for a more dense and impervious mix. On the other hand, excessive quantities of filler will significantly increase the asphalt demand, because of the increased surface area. Typically, the ratio of (filler)/ (aggregate + filler), by weight, have fallen in the range of 11 percent to 13 percent. It should be recognized, however, that workability is the key issue.

Most filler is derived from the dust of the crushed stone aggregate, supplemented, as necessary, by adding non-plastic fines, derived from limestone dust or Portland cement. Asbestos fibers have also been used in the past. They offer a structural advantage, but are no longer used, because of the health hazards they present to the workmen and the potential harm to the quality of the water. Inert fibers, such as polypropylene, may offer promise.

5.3.4 Asphalt Concrete Mixes

An asphalt concrete core, or each layer of an asphalt concrete facing, has a unique mixture of asphalt, aggregate and filler, designed to provide the intended function(s) of that element. Refer to Figure 5.2.

1. The core, or the impervious layers (I and I1) of a facing system, feature a well graded aggregate and filler mix, and generally from 6 to 8 percent asphalt, compacted to about a 3 percent air voids content.

2. The drainage layer (DR) uses an open graded coarse aggregate, with little to no

sand, no filler, and only enough asphalt (usually 2 percent to 4 percent) to thoroughly coat all of the aggregate surfaces. Additionally, this layer receives very light compaction, with the “break-down” pass (first roller pass) being delayed until the mix has cooled almost to the “stop-rolling” (last roller pass) temperature.

3. The aggregate mix for the base or bedding course or courses of a facing can vary from semi-open graded to uniformly graded, depending on how many of the functions (i.e., binding (BN), leveling (LV) or bearing (BS)) the particular layer is being designed to perform. The asphalt content varies accordingly; always being in sufficient amount to fully coat the aggregate and filler.

Special grades of asphalt or asphalt emulsion, varying in penetration from 100 to 40, are specified for the prime coat on the embankment, the tack coat between layers, the seal coat (SL) on the impervious layer, and the protective coating or layer (PT). Some designs forego the prime and/or tack coats, with reason.

The asphalt mastic seal coats often contain calcareous filler or mineral fibers, mixed on about a 1:1 basis with the asphalt. Sometimes sand has been used as the additive, in which cases the sand content was about 20 percent. Where asbestos fibers were added, their content was in the 2 to 6 percent range. The seal coat provides aging protection for the impervious layer and may need renewal.

The dense pack of the aggregate in the impervious layer, ranging from 2.1 to 2.5 tonnes/m^3, coupled with the relatively high asphalt content and a high degree of compaction, results in a mix that has about 3 percent voids, and is basically impervious. To reduce the voids much below 3 percent will lead to instability, due to asphalt pore pressures that lead to the shear strength becoming dominated by the viscosity of the asphalt rather than the intergranular friction of the aggregate. Depending on the functional design of the base course, its porosity will vary from 3 to 10 percent; and that of the drainage layer, will vary from 10 to 30 percent.

Permeability’s of the various layers vary from impervious (10^{-7} to 10^{-9} cm/sec, and lower) for the core or the impervious layer(s); to 10^{-2} cm/sec for the drainage layer. The bedding or base course of a facing will again vary, according to its designed function(s), from 10^{-2} cm/sec to impervious.

Other physical properties of the asphalt concrete mix such as stiffness, stability, durability, permanent deformation, resistance to stripping, fracture, workability etc. are important. Some of the important tests related to these properties are listed under Materials Testing.

Critical Temperatures

Prior to mixing, the aggregate must be dried to at least 0.5 percent moisture content and heated. The asphalt cement must also be heated, but not beyond the point that will drive off too many volatiles.

The mix must be stable on the slope when it is being placed. This is an important temperature related item and must be established by test during the design phase.

The break-down compaction pass for the impervious layers is normally made when the asphalt in the mix has a viscosity of between 1 and 10 poises. The generally accepted "stop rolling", i.e., the temperature lower than which additional roller passes will accomplish no further reduction in voids, occurs when the viscosity of the asphalt reaches 100 poises. When placing the pervious layers, the break down pass is deferred until almost the "stop rolling" temperature has been reached, so that the compaction will be light.

The critical temperatures depend on the viscosity of the asphalt. Table 5.1 shows their relationship.

Table 5.1. Critical Temperatures Related to Asphalt Concrete

Field Activity	Asphalt Viscosity (Poises)	Temperature Low Viscosity Asphalt °C (°F)	Temperature High Viscosity Asphalt °C (°F)
Mixing		165 (325)	178 (355)
Start Rolling	1 10	155 (310) 110 (225)	170 (340) 125 (255)
Stop Rolling	100	80 (175)	95 (200)

5.4 MATERIALS TESTING

Materials testing is an important element of both the design of an asphalt concrete core or facing for an embankment, or lining for a reservoir; and the quality control of its construction. The testing relates to the components of the mix (i.e., asphalt, aggregate and filler), and to the mix itself.

5.4.1 Design Tests

The purpose of these tests is to select and specify the most suitable materials and mix proportions for the core or facing being designed, recognizing the environmental and service conditions peculiar to the project.

5.4.2 Tests of Asphalt

These tests fall into four groupings: (1) **composition**; (2) **information for worker's safety**; (3) **consistency**; and (4) **durability**, as summarized on Table 5.2.

Table 5.2. Tests of Asphalt

Test	Description	ASTM
Composition Tests		
Solubility	Measures solubility of AC in CS_2 or trichloro-ethylene	D-2042
Specific Gravity		D-70
Loss and Drop in Penetration upon Heating	Heats a standard sample for 5 hrs. at 163°F, driving off volatiles and then, after cooling, measure the loss in weight and drop in penetration	D-6
Worker's Safety Information		
Pensky-Martens		D-93
Cleveland Open Cup	More commonly used procedure.	D-92
Consistency		
Ring and Ball	Tests for softening point	E-28
Penetration		D-5 D-946
Capillary Viscosity		D-2170
Cone and Plate viscometer		D-3205
Kinematic Viscosity		D-2170
Sliding plate microviscometer		D-3570
Vacuum capillary viscometer		D-2171
Ductility		D-113
Durability		
Thin film oven test	Tests that attempt to duplicate hardening that take place during mixing as a result of volatiles being driven off	D-1754
Rolling film oven test		D-2872

5.4.3 Tests of Aggregate

In physical respects, aggregates for asphalt concrete cores or facings should qualify as aggregates for Portland cement concrete. Tests for these aggregates have been standardized by the United States (ASTM), the UK (BS), Europe and Asia, and these respective standards have been recognized by the international community. The tests are listed on Table 5.3, by Standard, with no further explanation, since the standards are readily available for reference.

Table 5.3. Tests of Aggregate

Test	ASTM	BS
Specifications for Concrete Aggregates	C-33	
Aggregates from Natural Sources for Concrete		882
Methods for Sampling and Testing Mineral Aggregates, Sands and Fillers		812
Sampling, Shape, Size and Classification (BS 812)		Part 1
Physical Properties (BS 812)		Part 2
Mechanical Properties (BS 812)		Part 3
Chemical Properties (BS 812)		Part 4
Surface Moisture in Fine Aggregate	C-70	
Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfage	C-88	
Resistance to Abrasion of Small Size Coarse Aggregate by Use of the Los Angeles Machine	C-131	

5.4.4 Tests of Filler

It is important that the filler in the mix be non-plastic, and chemically compatible with both asphalt and water. The significant tests are gradation (hydrometer analysis) and the Atterberg Limits. Reference is made to Table 5.4.

Table 5.4. Tests of Filler

Test	ASTM	BS
Materials Finer than No. 200 Sieve in Meneral Aggregates by Washing	C-177	
Sampling and Testing of Mineral Aggregates, Sands and Fillers		812

5.4.5 Tests of Asphalt Concrete Mixes

Tests of asphalt concrete mixtures are grouped according to **qualitative** tests and **performance** tests. Those that are applicable to the design of asphalt concrete water barriers hydraulic structures are listed on Table 5.5.

Table 5.5. Tests of Asphalt Concrete Mixes

Test	ASTM
Qualitative Tests	
Asphalt Content by Nuclear Method	D-4125
Laboratory Reduction Method	D-4 D-2172
Compaction (kneading)	D-1561
Percent air voids	D-3203
Bulk specific gravity	D-1188
Degree of particle coating	D-2489
Performance Tests	
Flexibility	none
Stability	none
Compressive Strength	D-1074
Effect of water on cohesion	D-1075
Resistance to plastic flow (Marshall apparatus)	D-1559
Resistance to deformation (Hveem apparatus)	D-1560
Cohesion (Hveem apparatus)	D-1560
Triaxial Compression	D-2850
Diametral	C-496
Joint flow	None
Ball flow	None
Permeability	D-3637
Abrasion	None
Adhesion (stripping)	D-1664
Immersion-compression test	none

Most of the above tests on asphalt concrete mixtures have evolved into ASTM Standard Testing Procedures. In general, those that have will not be commented on here, because the ASTM Specifications speak for themselves. Those that are not ASTM Standards are generally Dutch Shell tests for asphaltic mixtures in hydraulic engineering, and merit reference (Ref. 5 and 6).

5.4.6 Frequency of Quality Control Testing

The frequency with which the various quality control tests are taken depends on the rate of production, and many other factors, including the engineer's judgment. Vigilance at the batch plant, supported by periodic testing, is a continuous activity, because consistency of the plant product is essential to maintaining a consistent high quality content of the in-place product. Then a consistent placing and rolling pattern (based on test pavements) will result in a dense, stable and impervious water barrier. Guidelines follow.

Tests on Components of the Mix

Asphalt.
Penetration index and softening point: 1 test each batch.

Aggregate and Filler.
Gradation: 1 test per week.
Density and voids content: 1 test per month.

Tests on the Mix at the Batch Plant

The batch plant (mixing plant) for the construction of either an asphalt core or an asphalt facing is exactly the same as that for a highway AC paving project. The quality control tests of the mix at the plant include:

Gradation, Asphalt Content, Density, Voids Content, Permeability and Stability on the Marshall compacted specimen: 1 to 2 tests, each, per day.

Temperature: Continuous monitoring.

Tests at the Construction Site

Flow stability on the slope: 1 test/ 4000-7000 m^2
Density, Voids Content, Permeability (on cored specimen):
Asphalt Concrete Core
1 test / two days
Asphalt Concrete Facing
1 test / 2150-4250 m^2 on Base Course (BS).
1 test / 5400-11500 m^2 on Impervious Layer (I)
In-Place Permeability Test on Impervious Layer (I):
1 test / 50000 m^2.

5.5 REFERENCES

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2. ICOLD Bulletin 32a (1982), *Bituminous Concrete Facings for Earth and Rockfill Dams*

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7th	Rome 1961	Question 27
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CHAPTER 6 — CONCRETE FACING FOR ROCKFILL DAMS

6.1 INTRODUCTION

Rockfill dams with upstream water barriers of wood have been used as early as 1850 by the gold miners in California. Later the wooden facing was replaced by concrete. Chatworth Park Dam in California constructed in 1895 was the first rockfill dam known to use concrete facing for the water barrier. The early use of concrete facing for water barriers was on dumped rock. Up to a height of 75 m they were satisfactory, but higher dams developed face cracks and excessive leakage, because of the high compressibility of the dumped rockfill. These experiences resulted in fewer rockfill dams with concrete facing being adopted. However, steep slopes of rockfill dams demonstrated the high shear strength of dumped rockfill and its usefulness as a dam building material. The high modulus of compressibility of compacted rockfill, observed in the high earth core rockfill dams, in addition to reviving the rockfill dams with concrete facing (CFRD), enabled small size rocks and rocks with low compressive strength to be used.

Important improvements in the design principles of rockfill dams with concrete facing adopted during the past 25-30 years have resulted in increased use of this type of rockfill dam and its adoption for higher dams. The advent of construction technology, through the use of properly zoned compacted rockfill and/or gravel, results in a dam of reliable performance in terms of safety and leakage. The development of concrete toe slabs with grouted cutoffs and face slab improvements, notably abandoning the highly articulated pattern of slabs and compressible joints, are the principal factors in current design trends that have resulted in the higher frequency of acceptance of CFRD.

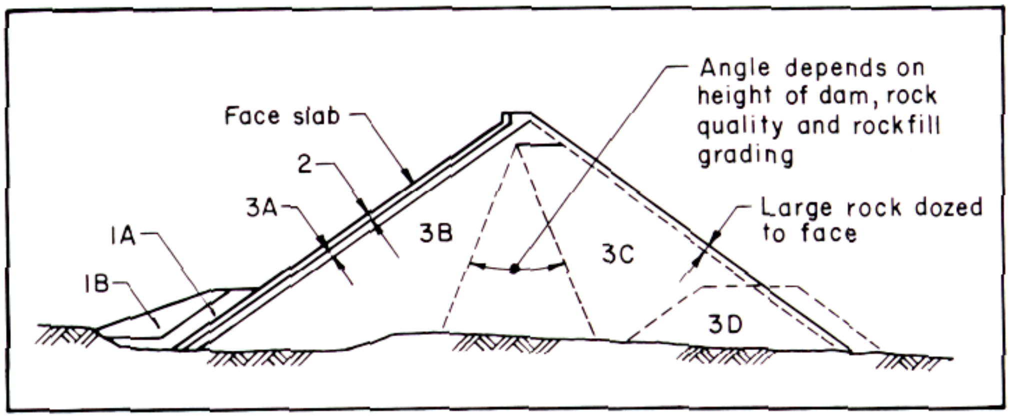
6.2 DESIGN

6.2.1 General

The design of CFRD has been mainly empirical and based on experience and judgement. The CFRD dams incorporating the design and construction changes leading to current practice have all performed well. Changes have been principally to effect economies in design and construction.

6.2.2 Typical Section

No type of dam is actually of standard design. Adaptation to the foundation and available materials, and consideration of each design element are necessary for each dam. However, the CFRD has evolved to a stage where the main elements are common. Figure 6.1 shows a typical cross section and zone designations for the CFRD of sound rockfill on bedrock.



Rockfill zones for Concrete Face Rockfill Dams of sound rock:

1A = Impervious soil

1B = Random 0.5 m layers

2 = Processed small rock transition

3A = Selected small rock placed in same layer thickness as zone 2

3B = Quarry-run rockfill, approximately 1 m layers

3C = Quarry-run rockfill, approximately 2 m layers

3D = Dumped rockfill. Compaction for rock zones: four passes of 10 t vibratory roller.

Figure 6.1. Typical Section of Rockfill Dam with Concrete Face

The concrete elements are similar in all dams: a toe slab (plinth), a monolithic face slab with joints only as necessary for construction, and a parapet wall. However, significant variations in fill, filter Zone 2 and rockfill Zone 3, have been allowed depending on the available construction materials.

6.3 MATERIALS

6.3.1 Types of Materials

The principal materials required for embankment dams with concrete facing as water barrier are **aggregates, cement** and **additives** for concrete; **earthfill** for upstream fill; **granular fill** for filters; **rockfill** or **gravel fill** for the main body of the embankment; **water stop** to seal the joints in the concrete slab; and **asphalt, shotcrete** or other materials for protection of the slope under the concrete face slab.

6.3.2 Aggregates

The requirements for both fine and coarse aggregates for use in concrete are that they be sound and free from organics and other deleterious materials. Durability of the concrete is more important than its strength; therefore, evaluation of soundness and reactivity of the aggregates is important. These evaluations are similar to the requirements for conventional concrete. Generally, the requirements of ASTM C33, Standard Specifications for Concrete Aggregates must be satisfied. Soundness of the aggregate can be tested by the method ASTM C88, Test Method for Soundness of Aggregate by use of Sodium Sulfate or Magnesium Sulfate. Alkali reactivity of aggregates is evaluated by two methods: ASTM C227, Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method) and ASTM C289, Test Method for Potential Alkali Reactivity of Aggregates (Chemical Method). The current practice is to limit the maximum size of aggregate to 1.5" (38 mm); however, a maximum size aggregate of 2.5" (68 mm) has sometimes been used and is satisfactory with special care taken at the construction and contraction joints, and water stops.

6.3.3 Cement

The cement used in the concrete is usually Portland cement Type II. However, if the aggregates indicate the need for high sulfate resistance, Type V cement is used. The type and quality of the cement is generally governed by the requirements of ASTM C150, Standard Specification for Portland Cement.

6.3.4 Additives

Generally, no accelerator additives are required in concrete used for water barrier facing. To enhance the durability and water tightness air entraining admixtures are used and the usual limit is 5 percent. Pozzolan, fly ash and plasticizer are used to reduce the water-cement ratio and to minimize the long term risk of alkali reactivity. It is considered good practice to use pozzolan or fly ash even with apparently non-reactive aggregates to provide a more impervious and durable concrete. Recently, use of silica fume has been recommended to improve the impermeability and thus reduce the potential of chemical attack of joints and reinforcement by reservoir water. Unless experienced and skilled workers are employed to properly mix and place concrete, use of silica fume is not recommended.

6.3.5 Concrete

As discussed above, durability and permeability are more important than strength; therefore, concrete of a 28-day compressive strength of 3000 psi (20 MPa) is adequate. On the basis of presently available experience and current practice, it is reasonable to use a face slab of constant thickness of 30 cm for dams of low to moderate height of 50 to 70 m and to use an incremental thickness of about 0.002 times the height for important and high dams. Where the reservoir drawdown exposes the concrete face for an extended period of time under extreme temperatures, polyfibers are sometimes used to control the development of shrinkage cracks in concrete; however, this is not a common practice. The face slab must be cured to minimize the effect of drying shrinkage. Normally, 28-day curing is sufficient, though longer curing time is sometimes specified.

6.3.6 Earthfill

Two types of earthfill, Zone 1A - nonplastic impervious fill and Zone 1B - random fill, are used in an upstream fill to cover the perimeter joint. The purpose of this feature is to seal any cracks or openings that might develop in the perimeter joint in the lower elevations of dams higher than 75 m. Zone 1A can be silty fine sand with 50 percent passing sieve #200. The material must be non-plastic so that under saturated condition it can move freely and seal the opening in the perimeter joint and thus reduce the potential leakage. The random fill can be silt, clay, sand and gravel that will cover and provide sufficient weight for the non-plastic material to enable it move into the opening. The combination of upstream fill and concrete face is used only in the lower part of the valley or canyon while a simple concrete face rockfill dam is adopted in the upper part of the valley.

6.3.7 Granular Fill

Two types of granular fill materials are used: Zone 2A - a fine filter and Zone 2B - a coarse filter. These materials can be either crushed or grizzlied small quarried rock or processed alluvial sand and gravel. Fine filter, Zone 2A, consists of sand with maximum particle size limited to about one inch and is used within a 3 m radius in plan from the perimeter joint to control the movement of the slab and thus to control the leakage through the joint opening. Coarse filter, Zone 2B is a minus 7.5 cm crusher-run rock placed underneath the face slab to provide support for it. Commonly used range of Zone 2B placed under the face slab is as follows:

Size of Opening	Percent passing by weight
3” (78 mm)	90 - 100
1-1/2” (38 mm)	70 - 95
3/4” (19 mm)	55 - 80
No. 4 Sieve	35 - 55
No. 30 Sieve	8 - 30
No. 200 Sieve	0 - 10

Fine filter, Zone 2A, will have similar gradation except that the maximum size is limited to one to one inch.

The several leakage incidents in CFRDs of compacted rockfill have been in or near the perimeter joint, and the filter allows convenient sealing by dirty fine sand (minus No. 10 or No. 20 mesh). There have been no crack or leakage events elsewhere in the face slab, and the more economical crusher-run zone provides a workable and dense zone on which to place the concrete face.

Zone 2A and 2B granular fill materials are placed in 30 to 40 cm layers and compacted by 4 passes of a 10 ton smooth drum vibratory roller. Such a compacted Zone 2A will have a coefficient of permeability of less than 1×10^{-3} cm/sec and thus eliminate the possibility of large leakage.

6.3.8 Rockfill ─ Zone 3

The majority of the construction material for the concrete face rockfill dam consists of rockfill. This rockfill zone is further divided into several subzones. The increased layer thickness from 3A to 3B to 3C and possibly to dumped rockfill, 3D, is to provide a high modulus where it is needed, and to provide an increase in permeability. The thicker downstream layer, 3C lowers construction costs and reduced tonnage of rockfill. The density of the 2 m layer of rockfill is about 7 per cent less than for the 1 m layer of rockfill, with consequent savings in rock volume. In several dams, dumped rockfill has been used in the downstream toe of the recent CFRD. It could be used more often, particularly, if no future raising of the dam is planned.

Dumped rockfill can be used in the downstream toe without affecting the face slab deformation. Underwater dumped rockfill has been successfully used in the downstream toe area to serve as cofferdam at Segredo and to permit rockfill placement prior to diversion at Xingo.

Weak rock is also used (Kangaroo Creek, Little Para, Mangrove Creek) with special consideration of the rock properties, placement procedures, zoning and drainage provisions. For foundation rock with permeable and possibly erodible features, the cut-off can be extended downstream from the toe slab (plinth) by shotcrete covered by filter material (Reece, Salvajina, Winneke).

6.4 COMPACTED ROCKFILL

6.4.1 General

Compacted rockfill properties that are particularly useful to the CFRD are high shear strength and a high modulus of compressibility. The high shear strength has been demonstrated by steep slopes of some existing CFRDs and by triaxial tests. The modulus of compressibility of compacted rockfill is 5 to 8 times higher than that of the dumped rockfill. The modulus of gravels, in general, is several times greater than that of compacted rockfill. Therefore, gravels may be used exclusively or in combination with rockfill for CFRD.

6.4.2 Placing

For sound rock, the dumping and spreading of rockfill is intentionally done to obtain segregation, see Figure 6.2. End dumping is on the edge of the layer being placed, and several passes of the dozer spread the rock. There is inherent segregation both in the dumping and in the spreading. The smooth surface of the previous layer is desirable. The large rocks tend to contact the smooth surface with a flat face. Scarifying or removal of fines is not required. The smooth surface of fines is desirable to reduce truck tire and dozer track costs. Horizontal permeability is much higher than vertical permeability.

6.4.3 Compaction

The traffic of loaded trucks and the movements of the spreading dozer provide effective compaction, which is supplemented by several passes of the smooth drum vibratory roller. For sound rock, the fines and small rocks in the upper zone of a layer are well compacted. In the lower zone, energy is effectively transmitted through large rocks, causing wedging and crushing of contact edges and points. For sound rock, compaction by four passes of a 10 t vibratory roller has become a standard practice. Commonly used layer thicknesses are 40-50 cm, 80-100 cm and 1.5 to 2.0 m for rockfill zone 3A, 3B and 3C, respectively.

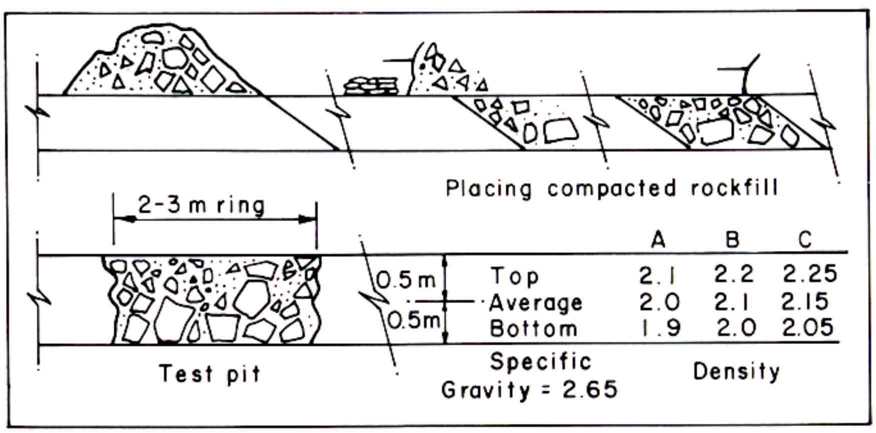


Figure 6.2. Placing and Density of Compacted Rockfill

Inadequate compaction near the perimeter joint has resulted in more face offset than desired. It is not possible to effectively compact the rockfill near the joint with the vibratory roller. Erratic and sometimes large offsets have been measured. Future offsets will be less, with the current specified use of a backhoe-mounted plate vibrator on the face and on the plan surface within 3 m of the perimeter joint. For weak rock, the factors of number of passes, layer thickness, and the use of water must all be considered.

6.4.4 Density

Density is of interest, but does not have the significance that it has in soil compaction. The influence of soil mechanics thinking and practice on rockfill placement has resulted in excessive waste and higher density, and in more restrictive specifications, than required. Density tests have been carried out on many dams, not as a control, but to learn about compacted rockfill and to have a record of the composition of the dam. For grading tests of sound rock, where gradings and densities have been taken in the upper and lower half of a 1 m layer, the upper, finer graded rockfill is about 8 percent more dense than the lower, coarse-graded rockfill.

6.4.5 Grading

Since all rockfill of sound rock is highly segregated in each layer, grading of the quarry-run rock is not important. Well-graded quarry-run rock will give the highest density and modulus, but all quarry-run rock, even when poorly graded, has been satisfactory. Strength in a layer of rockfill of sound rock comes from the density of the upper zone of fine rockfill and from the wedged and interlocked rocks in the lower zone. Foz do Areia is an example of an excellent CFRD of poorly graded quarry-run basalt.

For weak rock, grading is meaningless, and procedures (layer thickness, compaction, use of water) are selected to ensure breakdown and high density. Strength comes from the high density of the pieces of the rock in a dense matrix of fines from crushed rock.

6.4.6 Water

The use of water always improves rockfill properties, particularly in reducing compressibility. For rock having low water absorption (less than about 2 percent) the benefit is small and the use of water may only be justified in selected zones of high dams. For low compressive strength rock with high water absorption, the loss of strength on saturation can be 40 to 60 percent and the use of water requires serious consideration even for low dams.

A rockfill condition for hard rock, that may require water during placement, is the excessive presence of fines (minus no. 200 mesh). Enforcing a specified maximum percentage is not practicable. If there is an appearance of excessive fines, a check can be made by saturating the area with a water wagon and then observing whether a loaded dump truck and the vibratory roller are supported. For rock from a source known to contain a high percentage of fines, the application of water during placement may be specified.

6.4.7 Shear Strength

Early California dumped rockfill dams of 80 to 130 ft (25 to 40 m) height had steep slopes and demonstrated shear strengths of the order of 60E even with high field sample void ratios. A ϕ value of 45E can be assumed for compacted rockfill of sound rock. However, since there have been no slope stability failures of the CFRDs of dumped or compacted rockfill, slopes have generally been based on precedent rather than stability analyses.

6.4.8 Modulus

The vertical modulus of compressibility during construction is an indication of rockfill quality. It is obtained by water level devices or crossarms. The modulus determined from face deflection measurements is 1.5 to 3 times higher. Values of vertical moduli obtained in earth core rockfill dams and concrete face rockfill dams have ranged from 4,500 - 19,500 psi (30 to 130 MPa), depending on the rock, rockfill grading, layer thickness and other factors. The usefulness of moduli data is principally for the highest dams and future higher CFRD dams.

6.5 REINFORCING

The main purpose of the reinforcing is to function as temperature steel, and to spread out and minimize the widths of any cracks. In toe slabs reinforcing and dowels are useful as a grout cap. A single layer should be used. The steel is put 4 to 6 in (10-15 cm) clear of the upper surface as temperature steel, where it is hooked by the anchors. A double row of longitudinal steel has a slight theoretical disadvantage because it makes the toe slab stiff and less able to adjust to any small differential settlements of the underlying rock, which has no tensile properties. Anchors in the toe slab are used simply to pin the concrete to the rock. The anchors are not to resist any given uplift loads. Lengths, spacing, and bar diameters should be chosen on the basis of precedent and the characteristics of the rock foundation. The anchors and temperature reinforcing do improve the slab as a grout cap. Anchors used in common practice have generally been #8-#11, (25-35-mm-diameter) bars spaced about 4 to 5 ft (1.2-1.5 m) each direction, with lengths usually of 10-15 ft (3.0-4.6 m). The anchors are simple dowels of reinforcing steel, grouted full length in the rock, and hooked (90E) on the one layer of reinforcing. For face slabs the use of reinforcing of 0.4 percent of slab thickness in each direction, for compacted rockfill, has been an economical and successful change from the traditional 0.5 percent used with dumped-rockfill dams. Current practice is to use 0.3 percent steel in the large central area of known compression, and 0.4 percent near the perimeter and in the starter slabs. The steel area should be calculated on the basis of the design concrete thickness. A trend, which appears to be desirable and economical, is to carry horizontal reinforcing through the vertical joints. The trend is based on the fact that the major area of the face is under compression. Several vertical joints near abutments are contraction joints to minimize perimeter joint opening. Where reinforcing passes through vertical joints, a bottom waterstop has sometimes also been used as a carryover from the earlier practice. With the steel passing through, there is little or no more tendency for a crack to open at the construction joint than at other locations in the slab. Bonded joints are assumed in reinforced concrete design.

6.6 WATERSTOPS

Waterstops are used at the vertical contraction and construction joints, perimeter joints and sometimes at the toe slab joints to control seepage through the joints. Material types that have been used in waterstops in the already constructed and operating projects include stainless steel, copper, rubber, hypalon and PVC. The current trend is to use copper and PVC waterstops unless the reservoir water chemistry requires that a special corrosion resistant type waterstop be used. In such case, stainless steel and/or hypalon waterstops are used.

6.7 PROTECTION OF ZONE 2

After placing, trimming and compacting the Zone 2 material, the compacted surface needs to be protected against damage from erosion and construction activities until the face slab concrete is poured. Such protection is often obtained by applying a layer of quick curing asphaltic emulsion at the rate of 2 to 4 liters/m^2 or about 4 to 5 cm thick shotcrete on the compacted Zone 2 surface.

6.8 REFERENCES

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CHAPTER 7 — GEOSYNTHETICS

7.1 INTRODUCTION

The past 50 years has seen the use of synthetic products in civil works gradually increase to become commonplace. The development in the 1950s and 1960s of synthetic materials resistant to biological or environmental degradation opened the way to their use in permanent structures such as dams. With the development of new and improved materials came new technology for testing, specifying, and evaluating these materials, as well as new terminology. Initially materials were referred to in terms of their composition, such as plastic linings or reinforced bituminous fabrics. Later generic terms began to be used such as flexible membrane linings. The appreciation of the role of synthetic materials eventually led to the current term geosynthetics.

Use of geosynthetics in dams in general has lagged behind their use in other civil works because the dam engineering community has been slow to accept geosynthetics due to the lack of long-term performance data and the conservative nature of the industry. As a result, it is generally accepted that geosynthetics in dams should be located where they can be replaced, if necessary, and should not be used in applications where they are the sole defense against dam failure (i.e., where failure of the geosynthetic could lead to catastrophic failure of the dam).

The first uses of geosynthetics in dams were for special applications in embankment dams. The first major use of a geomembrane in dam construction was in 1960 by Karl Terzaghi who used nearly 10,000 m^2 of polyvinyl chloride (PVC) as the principal water barrier in the dam in Canada that now bears his name. In 1970, a geotextile was used in the Valcros Dam, in France, as a filter in two different locations.

In the last 25 years there have been major developments in new and improved materials. Along with the development of new materials came the establishment of standards for the materials used and general engineering principals for their use. This chapter addresses the use of geosynthetics in dam design and construction. It summarizes the two ICOLD bulletins *Geotextiles as Filters and Transitions in Fill Dams* (ICOLD 1986) and *Watertight Geomembranes for Dams, State of the Art* (ICOLD 1991), as well as additional information known to the committee. Although these two volumes are becoming dated, they remain excellent references on the use of geosynthetics for dams. This chapter is intended to provide an overview of current information on geosynthetics for dams. For detailed design, the reader is advised to consult the documents listed in the Bibliography of this chapter and current technical literature since the properties and uses of geosynthetics for dams continues to advance.

7.2 GEOSYNTHETIC MATERIALS

The term geosynthetics includes both geotextiles and geomembranes. These two broad categories are differentiated by their permeability. Geotextiles are commonly considered to be permeable fabrics, while geomembranes are commonly defined as low-permeability waterbarriers.

7.2.1 Geotextiles

Geotextiles are porous fabrics made of a synthetic fiber. They are typically manufactured and delivered in rolls. Adjacent strips are commonly overlapped, though they are occasionally sewn together. Geotextiles are predominantly manufactured from polyester and polypropylene; other materials that are used include nylon, polyethylene, polyvinyl chloride and fiberglass. Two major types of geotextiles are commonly manufacturing depending on the method used to bond the individual filaments to form fabric:

1. Woven fabrics with heat-bonded or mechanically bonded fibers; and
2. Non-woven fabrics with staple or continuous filaments that are needle-punched, heat bonded, resin-bonded, or bonded with a combination of these methods.

7.2.2 Geomembranes

Geomembranes are low-permeability water barriers. They are commonly manufactured and delivered in rolls and seamed on site. Some geomembranes have been formed in large panels before installation and delivered to the site for installation. Geomembranes are manufactured from synthetic polymers and/or bituminous materials. The synthetic polymers can be categorized in two broad groups: thermoplastics and elastomers.

Thermoplastics typically exhibit plastic stress-strain behavior. They can be thermal readily welded with hot air, hot wedges, or extrusion. Some have high coefficients of thermal expansion and expand or shrink depending on ambient temperature changes. Others have reduced flexibility in cold temperatures. Common thermoplastics are:

- Polyvinyl chloride (PVC) in several variations
- Low-density polyethylene (LDPE) and high-density polyethylene (HDPE)

PVC is susceptible to loss of flexibility from heat, but can be treated to resist deterioration to sunlight (UV radiation), and generally has satisfactory resistance to a wide range of chemicals. HDPE exhibits good chemical stability when exposed to heat, sunlight, and chemicals. However, it tends to be stiff and have a high coefficient of thermal expansion that complicates installation. LDPE exhibits similar stability as HDPE, but is more flexible.

Elasotomers typically exhibit elastic stress strain behavior. Seaming varies depending on the material and may include thermal welding, welding, solvent, adhesives, vulcanization, or tape. These materials tend to remain flexible at low temperatures and to have lower coefficients of thermal expansion. Common elastomers are:

- Isoprene-isobutylene rubber (IIR or Butyl Rubber)
- Ethylene-propylene diene monomer (EPDM)
- Chlorinated polyethylene (CPE),
- Chlorosulfonated polyethylene (CSPE or Hypalon)

Butyl rubber gradually perishes from exposure to ozone and is highly sensitive to hydrocarbons. CPE has good chemical stability to heat, sunlight and some chemicals. It has a tendency to adsorb water at higher temperatures. CSPE has similar performance as HDPE, but repairs tend to be more difficult with time.

Bituminous materials can be categorized in two broad groups - oxidized bitumen and bitumen elastomer. Both exhibit visco-plastic stress strain behavior and can be readily thermal welded. Oxidized bitumen is significantly affected by high temperatures and has reduced flexibility at low temperatures, whereas the effects are less pronounced in the bitumen elastomers. Bituminous geomembranes are highly resistant to ultraviolet radiation, although the surface may craze if there is no protective layer.

7.2.3 Other Geosynthetics

Other geosynthetics have been used for special engineering applications in civil works including dams:

1. Geogrid — Comparatively open-mesh synthetic grids used principally as tensile reinforcement of soil.
2. Geonet — Very porous net-like synthetic material used to permit high water transmission in the plane of the material.
3. Geocomposite — Specialty materials that combine two or more geosynthetics, such as a geomembrane combined with a geotextile for higher friction and puncture resistance in a watertight barrier. Another is a geotextile combined with a geonet for strip or wick drains.

7.2.4 Testing

Testing for geosynthetics can be classified into:

- Quality control testing during manufacture
- Identification/acceptance testing
- Performance testing

Manufacturers perform quality testing to control the quality of the finished product including such things as nominal weight per unit area and tolerance; nominal thickness and tolerance; roll width, length, and diameter. This information is included in the product specification.

Manufacturers also perform identification/acceptance testing of the geosynthetic. These tests enable easy verification by the purchaser and usually consist of chemical composition tests, physical/chemical property tests, mechanical/rheological property tests, and thermal and thermomechanical properties. The pertinent physical test results are also included in the product specification.

Performance tests provide the additional information needed to select a geosynthetic for a particular application. These tests usually evaluate engineering properties and durability. There are a myriad of tests for evaluating geosynthetics and the reader is referred to the references for more resources. Geosynthetic performance tests commonly include:

- Uniaxial tensile strength
- Grab strength

- Seam peel and shear strength
- Tear strength
- Puncture strength
- Burst strength
- Impact strength
- Permittivity (permeability/thickness)
- Apparent opening size (geotextile only)
- Coefficient of friction between geosynthetic and soil or other material
- Environmental tests to evaluate geosynthetic behavior with respect to ultraviolet radiation from the sun, temperature, resistance to chemicals, and microorganisms.

7.3 ENGINEERING CONSIDERATIONS

Unlike natural materials that have to be found near a dam site in suitable quantities and acceptable qualities to which the design is adapted, geosynthetics can to selected or manufactured to meet a given set of specifications. Geosynthetics are available in a wide range of engineering properties that address requirements for specific applications as a water barriers, filters, separation, tensile reinforcement, drainage, protection from mechanical damage, and erosion control. Geosynthetics generally offer a rapid means of construction and are often competitive or lower cost when compared with other methods and materials. Geosynthetics have properties that often offer solutions to situations that are not solvable with natural materials.

Geosynthetics can be used as both temporary construction expedients and permanent dam components. Temporary uses during construction that are not critical to the primary function of the dam include reinforcement/drainage for temporary haul roads, temporary spillway erosion protection, drainage of fill to speed consolidation, constraining filter or drainage media to desired profiles and primary water barriers for cofferdams.

This chapter focuses primarily on permanent uses of geosynthetics for dams. Permanent uses can be divided into those for which replacement is practicable and those for which replacement is impracticable. Practicable replacement generally requires the geosynthetic to be near the surface of the dam, such as a component of upstream or downstream slope protection, crest roads, toe drain wrappings, and upstream waterbarriers. Applications of geosynthetics that typically would not be replaceable include reinforcement and drainage in permanent roads or embankments, soil reinforcement in dam embankments, separation of materials and filtration, and internal drains or internal water barriers. Although geosynthetics have been used within the bodies of dams, where monitoring and replacement is impracticable, this has generally been limited to smaller, less critical structures.

Geotextiles may be used for several functions such as separation, filtration, drainage, and reinforcement, whereas geomembranes are typically used only as water barriers. The proper geosynthetic must be selected for the application. Its properties must be detailed in the materials specifications so that the particular geosynthetic delivered to the job will have known characteristics. The specifications should include the required physical, mechanical, chemical properties, and engineering properties of the geosynthetic.

As with all engineering materials, geosynthetics have limitations. Many problems in works engineered with geosynthetics come from not understanding these limitations. For example, geomembranes are to form a watertight barrier and not to provide a structural element. The major structural stress the geomembrane should encounter is during its placement. From then on, it should be structurally supported so that it can perform its intended function of a watertight barrier.

The durability of geosynthetics when properly manufactured and handled is good. However, particular environments exist for which each material should be avoided in specific applications, or during construction. Some materials, for example, are impact sensitive - dropping cover material with large sharp stones can lead to permanent damage. Other materials should not be used in an environment exposed to the elements, particularly to the sun. In such instances, a design that does not call for protecting the geosynthetic will result in poor performance. Some materials have a large coefficient of thermal expansion that can cause problems during construction when proper procedures are not taken. It is necessary to not only understand the advantages of the geosynthetics and use them appropriately, but also to be aware of their limitations and assure that they are considered in the design of the project and during construction.

7.3.1 Mechanical Loads

Geosynthetic materials are subjected to mechanical loads and physical, chemical and biological attack. Mechanical loads are caused by: handling during installation, sliding, differential subgrade deformation, punctures, tensile stresses, impact, wind, waves, ice, uplift from water or gas, and expansion.

Punctures and tears can occur during installation of the geosynthetic, while placing material over the geosynthetic, and after the dam is placed in operation. Punctures and tears during installation can be avoided by careful subgrade preparation and construction methods.

Geosynthetic materials may create a plane of preferential slip; therefore, stability against sliding must be checked at every interface. If sliding occurs on any interface, there is the risk of tearing the geosynthetic layers. Tests by (Martin 1984) show that interfaces between geosynthetics and granular soils have lower shear strength than that of the adjacent soil. They also report test results of interfaces between geomembranes and geotextiles, which are rather low ***(friction angles on the order of 6 to 28 degrees, depending on the materials)***.

Differential subgrade deformation chiefly concerns the abutment and foundation interfaces where there is a change in stiffness, sudden changes in the slopes of the abutment, interfaces with concrete structures, and areas at the top of the dam where thin layers may be anchored in the fill. Care must be taken at each of these to avoid overstressing geosynthetic materials.

Tensile stresses may be induced in geosynthetics by the effects of placing, spreading, and compacting material over a geosynthetic. If high tensile stresses are expected, then a reinforced geosynthetic such as a geomembrane bonded to a geotextile may be appropriate.

Impact loads from dropped tools and construction equipment during construction and floating debris, wildlife, and vandalism after the dam is put into service, can cause punctures, or tears in geosynthetics. Careful control of the construction and the use of an overlying protective layer and underlying supporting layer is often the best protection.

Wind can interfere with placement of the geosynthetic. Wind can lift a geosynthetic once it is installed and prior to placement of an overlying protective layer. Typically, geosynthetics are weighted down with sand bags during construction to counteract the negative pressure due to the wind.

Waves can displace protective layers and fatigue exposed geosynthetics from repeated deformation.

For the installation of geomembranes on the upstream surfaces of dams, the following additional considerations are appropriate:

- The best protection is a heavy protective layer designed to resist wave loads. Waves will form a bench in the underlying materials on unprotected membranes and, over time, may cause them to rip or tear.
- Floating ice on the surface of the reservoir can induce tension in the geomembrane and lead to rips or tears. Bubbler and propeller systems may be necessary to prevent the formation of ice adjacent to the geomembrane.
- Uplift may develop from seepage during rapid drawdown conditions if the underlying material is saturated (e.g., from high tailwater, groundwater, or leakage). The uplift pressures will tension the geomembrane and form blisters or bulges. Proper drainage beneath the geomembrane is the best protection against uplift pressures. If the geomembrane extends over the floor of the reservoir, decomposing organic mater may produce gas that could be trapped beneath the geomembrane. In these situations, provisions to collect and vent the gasses should be considered.

HDPE has a high coefficient of thermal expansion and is therefore alternately stretched and slackened in hot climates, causing ripples and tensile stresses at seams and connections to concrete structures. The effect is especially prevalent where the HDPE does not have a protective layer and the reservoir level is fluctuated so that the geomembrane is exposed the ambient air changes. In some cases, HDPE has been installed at night or cool weather to avoid locking in tensile stresses.

Although minor damage to a geotextile may be acceptable if it is used in a temporary application or for separation, geotextiles used as filters must be installed carefully to avoid damage. Precautions that can be taken to minimize damage during installation include:

- Preparing the subgrade so that depressions or humps above level are limited to less than 1 foot in 10 feet
- Avoiding equipment traffic on unprotected geotextile
- Covering the geotextile with a layer of sand or gravel
- Including a compacted base layer of sand or gravel beneath a geotextile
- Limiting the height of drop when placing rip rap on geotextiles

7.3.2 Physical, Chemical and Biological Attack

Geosynthetics are subjected to a variety of environmental loads such as heat, ultra violet radiation from sunlight, pollutants, microorganisms, vegetation and rodents. Ambient temperature change and the coefficient of thermal expansion must be considered when selecting a geomembrane. Resistance to damage from ultraviolet radiation can be lessened by proper formulation of the geosynthetic and storing it out of direct sunlight. Pollutants in soil and water, such as hydrocarbons, can deteriorate geosynthetics and geomembrane seaming systems. Solvents and adhesives should be tested for sensitivity to potential contaminants. Aggressive microorganisms may exist in the fill and foundation or develop in the reservoir and should be considered. Roots in soil beneath geosynthetics can grow through and puncture them. Herbicides may need to be applied to the subgrade, but should be checked for compatibility with the geosynthetic. Rodent attack of a geosynthetic may occur when a rodent is trapped beneath a geosynthetic or it is searching food is buried beneath it.

7.4. GEOMEMBRANES FOR EARTH DAM WATER BARRIERS

Geomembranes are becoming more common for upstream water barriers for both earth and concrete dams, often in combination with geotextiles. Typical applications are:

- Linings for water reservoirs retained by earth dams
- Facings for earth water storage dams
- Repair of earth dams that have water barriers that have deteriorated or are inadequate.
- Water barriers on the upstream face of new or existing roller compacted concrete (RCC) dams
- Water barriers on the upstream face of existing concrete dams to reduce water seepage

Geomembranes for upstream water barriers in earth dams are composite layered systems that typically include a protective layer, geomembrane, supporting layer, and base layer over the body of the dam. Each is discussed below.

7.4.1 Dam Embankment Subgrade

For embankment dams, the stability of the geomembrane on the upstream slope depends on the frictional resistance between the geomembrane and the underlying dam embankment subgrade material. Typical slopes for different types of subgrade materials are:

Material	**Typical Slope** **(Horizontal:Vertical)**
Clay	2.5:1 to 3.5:1
Sandy clay and silt	2.0:1 to 3.0:1
Sand and gravel	2.0:1 to 2.5:1
Rockfill	1.5:1 to 2.0:1

The 1.5:1 slope for rockfill is considered to be an upper limit where geomembranes are used. Although steeper slopes may be feasible in terms of dam stability, they complicate laying the geomembrane and placing the protective layer and hence increase construction costs.

Consistent with the philosophy that geosynthetics should not be the primary defense against dam failure, the stability of the embankment slopes should be analyzed for the all load combinations as if the geomembrane did not exist. This is especially pertinent for the rapid drawdown case where the material underneath the geomembrane could be saturated from leakage through the geomembrane.

Differential deformation between the upstream face fill and intake and outlet structures may be large and induce tension in the geomembrane. This problem can be difficult to solve for large deformations and relocation of the structures may be appropriate.

The upstream slope should be planar to facilitate geomembrane placement. A curved dam axis (upstream radius) can encourage the geomembrane to lift off the surface. Benches are acceptable; however, the geomembrane may need to be anchored at the upstream edge.

Since there is the risk of the geomembrane leaking as the result of damage or deterioration, it is essential to provide underdrainage, except in free draining rockfill embankments. For less pervious embankment materials, drainage is needed to protect the geomembrane from damage by uplift and to protect the embankment from piping. The supporting layer should be a free draining material discharging the collected water to the downstream side of the dam. If public safety is an important matter, the designer should consider the possibility that a large tear could cause flow in excess of the drain capacity and include a second drainage zone in the embankment. If the embankment material is semi-pervious material, an underdrain may not be necessary; but the fill should be drained with a downstream drainage zone.

7.4.2 Base Layer

The base layer provides a transition from the body of the embankment dam to the supporting layer. There are differing considerations for rockfill and earthfill dams. For rockfill dams, which are highly pervious, the base layer provides a transition in grain size from the dam to the supporting layer. In earthfill dams, the supporting layer must also act as a drain and must therefore the base layer should be designed as a filter to prevent fines from the body of the dam migrating into the drain material. A geotextile, if it offers the same performance, can be substituted for the filter material. In exceptional cases, a second geomembrane may be used with the drainage material sandwiched between the two membranes.

7.4.3 Supporting Layer

The supporting layer provides a transition from the base layer to the geomembrane. There are differing considerations for rockfill and earth dams. In rockfill dams, the supporting layer may consist of a fine, stabilized material such as bitumen, cement stabilized sand, lean concrete, or no-fines bituminous concrete. Alternatively, the supporting layer may consist of simply a thick, strong, geotextile laid directly on the base layer.

In earthfill dams, the purpose of the supporting layer is to drain leakage through the facing to prevent the build-up of uplift pressures. In most cases the drain is made of natural materials with a permeability of 10^{-2} to 10^{-3} m/s. Typical materials are: a) coarse sand or crusher run sand free of fines, at least 15 cm thick, spread and compacted along the slope; or b) open, free-draining bitumen premix at least 10 cm thick. The layer thickness must be appropriate to the height of the dam (i.e., anticipated amount of leakage). A geotextile over the drainage layer may be appropriate if there is danger of angular material puncturing the geomembrane. A geomembrane/geotextile composite may be used. Laying perforated pipes along the slope has been used to increase the drainage capacity of the layer.

The drainage layer should discharge to a finger drain through the dam or other drain system that is designed to avoid piping or clogging. The drain system should have multiple outlets to help identify the location of leaks.

In some cases, the natural material may be replaced by a thick geotextile acting as a filter at the surface and a drain in the middle. The drainage capacity of the geotextile must be adequate under the compressive stresses applied to it. The drainage capacity of geotextiles decreases toward the bottom of the slope due to the higher hydrostatic load and with current materials is not recommended for dams greater than about 15m high.

If the dam body is comprised of lower permeability material, it may not be necessary to include a drainage layer immediately beneath the membrane, rather it may be acceptable to drain the fill with a conventional internal drainage zone. In this case, the base and supporting layers are one and the same layer and may be replaced by a geotextile. In this application, the membrane may need to be buried to counteract uplift.

7.4.4 Geomembrane

If protective layer is not included in the design, the geomembrane should be permanently anchored in a trench at the top, once it has been installed and seamed. For designs with protective layers, top anchorage may be provided in a similar fashion once the protective layer has been installed. Alternatively, when a protective layer is provided, the top end of the geomembrane may be anchored by placing it along a horizontal lift in the fill.

The geomembrane is typically connected to the foundation by a special concrete or clay structure at the toe of the dam. For earth foundations, the geomembrane can be extended into a trench at the toe of the dam and sandwiched between compacted layers of suitable soils. For rock foundations, the geomembrane is usually fastened to a concrete block by clamping with a metal strip or welding to an embedded strip. Alternatively, the geomembrane can be extended upstream to form a seepage blanket, or in small reservoirs to cover the entire floor.

Geomembranes are connected to concrete outlet structures and spillways by clamping with a metal strip or welding to an embedded strip. Fill beneath the geomembrane adjacent to the concrete structures should be well compacted to avoid settlement that would stretch and possibly tear the geomembrane.

Geomembrane panels are usually connected with seams using solvents, adhesives, or welding methods to suit the particular material. Seams are often the weakest part of the facing and special attention should be given to their construction. The overlap for seams ranges typically ranges from 5 to 20 cm depending on the particular material and seaming method. Samples of seams should be taken for tensile and joint peel strength. Some thermoplastic materials are suitable for joining with a double seam that allows testing with air or water pumped in the channel between the seams. Sometimes, sliding seams are appropriate to allow movement between panels. These seams must have a deformable material between the panels to avoid tearing the geomembrane.

Rolled geomembranes are installed by positioning the rolls on the crest of the dam, overlapping the previously placed roll, and unrolling it from top to bottom. Folds and wrinkles should be avoided as they can weaken the geomembrane. The unrolled geomembrane must be weighted down to prevent it from being lifted by wind. Horizontal seams should be avoided due the potential of tearing under imposed tensile stresses.

7.4.5 Protective Layer

Protective layers may consist of rockfill, bituminous concrete, or prefabricated concrete blocks. The size and unit weight of rockfill protection is selected to resist the design waves. It must be placed without puncturing the geomembrane, which typically requires a protective layer such as a geotextile or properly graded gravel layer. An open-graded layer of bituminous concrete may be used for a protective layer if it is compatible with the geomembrane. It must be thick enough to resist waves and provide a means to dissipate uplift. Bituminous concrete can be placed hot or cold. A geotextile can be used to protect the geomembrane against puncture from placement of the bituminous concrete. Concrete blocks are usually tied together with interlocks or cables to resist wave action. They should be bedded on a properly graded drainage layer to avoid damage from uplift pressures.

On small dams, the protective layer has sometimes been omitted and the geomembrane has been protected from fatigue caused by repeated movement from wave and wind loads by a pattern of weights laid on the membrane or a pattern of concrete beams. Alternatively, the geomembrane can be adhered to a rigid backing.

7.4.6 Quality Control and Performance Monitoring

Stringent testing and inspection are required due to the relative fragile nature of geomembranes. During construction, the focus is on preventing damage to the membrane and proper construction of seams. One hundred percent testing of seams is generally appropriate. During first filling, frequent visual inspection and monitoring of leakage rates care should be done as the reservoir level rises. During operation, leakage rates should be recorded at more frequent rates than for dams without upstream membranes to allow early identification and repair of leaks.

7.4.7 Repair of Earth Dam Concrete Facings

Deteriorated concrete facings of earth dams have been repaired using geomembranes, using both *in situ* fabricated membranes and prefabricated membranes. *In situ* membranes have been fabricated by covering the face with a non-woven geotextile and applying acrylic monomer or bitumen latex to impregnate the geotextile, seal the panels together, create a watertight membrane, and seal the membrane to the underlying facing. This method eliminates the need for seaming, but has the disadvantage of having less control over the uniformity and thickness of the finished product.
Prefabricated membranes have been fixed to facings using adhesives or hot bitumen or a regular pattern of steel ribs.

The concrete facing must be cleaned and sharp points or edges removed. If the surface is rough a geotextile may be necessary to protect the membrane. Generally, crest anchorage is not sufficient since an overlying protective layer is not used. The membrane must be held in place over its entire area to prevent it from being lifted by wind or waves. Providing that the facing to be repaired and the dam are pervious, drainage is not required. Geotextiles and geodrains have been used to provide drainage.

Where the geomembrane is used to repair a concrete face, it may be acceptable to omit the protective covering since the concrete face provides a secondary water barrier. The uncovered membrane has the advantage of being easier to inspect.

7.5 GEOMEMBRANES FOR CONCRETE DAM WATER BARRIERS

Geomembranes are becoming common for repairing deteriorating concrete dams and to form the water barrier in new roller compacted concrete dams.

For repairing existing dams, the geomembrane is typically fixed to the upstream face by a grid of steel ribs. The ribs and geomembrane are anchored to the dam with steel bolts. A cover strip welded across the ribs seals the anchor boltholes. Drainage is typically provided via hollow ribs, geonet, or geodrain that directs seepage through the dam to a gallery or the downstream face. The geomembrane is typically exposed to the sunlight, wave, and ice action. PVC is commonly used, but other types have also been installed.

While a geomembrane would not be required for a new conventional concrete dam, they are sometimes used as water barriers for new RCC dams. The geomembrane is typically incorporated into the upstream face. Including the geomembrane integral with precast, interlocking concrete facing panels is becoming more common. The precast panels serve as permanent formwork for the RCC and improve the appearance of the dam. The geomembrane is fixed to the downstream side of the panels and is seamed with adjacent panels during construction. Penetrations for panel anchors into the RCC required special attention.

Recently, several new RCC dams have included upstream geomembranes on the upstream face that were installed following construction of the dam, similar to what has been done for repair of existing dams. In both applications, drainage behind the geomembrane is included to prevent buildup of pressure behind the membrane. Both a gravel layers and pipes that lead to a gallery or the downstream face have been used for drainage.

7.6 GEOTEXTILES AS FILTERS IN FILL DAMS

The permanent function of a geotextile in an embankment dam is likely to involve filtration. The extent to which filter performance is critical to the safety of the dam can be related to the nature and duration of flow against which protection is required, the extent to which failure of the filter is critical to the safety of the structure, and the practicability of repair if failure occurs.

Conservatism in design is essential and experience should be interpreted with caution. Each use of a geotextile should be evaluated individually and subjected to the judgment of the designer. Care must be taken that case histories of satisfactory performance of geotextiles in non-critical applications are not used to justify uses in critical applications. For example, successful use of geotextiles at interfaces where hydraulic stresses are low or the interface may have been stable without the geotextile do not demonstrate suitability for interfaces subjected to severe flow. Generally, the performance of the geotextile cannot be monitored *in situ* directly and evidence of deterioration may not be visible until considerable damage has occurred. Considerable caution is required in the design of transitions that are subjected to continuous seepage.

7.6.1 Possible Applications of Geotextiles as Filters in Earth Dams

Possible applications of geotextiles as filters are summarized in Table 7.1.

Table 7.1. Possible Applications of Geotextiles as Filters in Dams

Filter Location	Filter Purpose	Flow	Significance of Failure	Access for Repair
Downstream slope protection	Control erosion by rainfall	Occasional surface flow	Non-critical	Easy
Downstream surface drains	Removal of surface seepage	Continuous local seepage	Non-critical	Easy
Upstream slope protection	Control of erosion by wave action	Cyclic flow during wave action and small flow during drawdown	Usually not catastrophic	Possible
Temporary internal drainage	Dissipation of excess pore pressure during construction of wet fills	Temporary flow, limited quantity.	Non-catastrophic. Failure may lead to instability during construction or delays.	None
Upstream internal fill boundary	Prevention of migration of fines in an upstream direction	Transient and small flows during drawdown	Non-catastrophic. Only significant if migration is large and continuous	None
Downstream internal interface-no continuous flow	Prevention of migration of fines in a downstream direction	Flow due only to rainfall	Limited and non-catastrophic	None to possible with reservoir drawdown
Downstream internal fill boundary-continuous flow	Prevention of internal erosion, including effects of concentrated flow in cracks	Continuous flow from reservoir	Potentially catastrophic and rapid.	Generally none. May be possible with reservoir drawdown

7.6.2 Principles of Filtration

Filtration is the establishment of a stable interface between a fine soil (base) and a coarse soil or geotextile (filter), when the interface is subjected to flow from the base soil to the filter sufficient to cause particle migration. Factors to be considered during design include:

- Cohesion of base soil
- Density of the base and filter soil
- Potential for reversal of flow
- Potential for open cracks in base soil
- Internal stability of the base soil

For non-cohesive base soils, fine particles from a zone adjacent to the interface are moved by flow and arrested at the interface, plugging the pores of the filter by progressively smaller particles in a process known as self-filtering. The amount of particle movement will depend on the grading of the base soil and the pore distribution of the filter, which is a function of compaction and stress. For a properly designed filter, this process will lead to a stable interface. Good compaction (high densities and normal stresses) improves filtration. However, since it is difficult to ensure good compaction everywhere along the interface during construction, filter design should not be based on the assumption that high densities and high normal stresses will always exist.

Reversal of flow across an interface can inhibit self filtering and caution is required when designing filters for this situation.

Interparticle forces in cohesive soils produce a small tensile strength in terms of effective stress and allow the soil to bridge over the pores of a filter. Therefore, rules developed for non-cohesive soils do not apply to cohesive soils. A filter with larger pores than predicted by such rules can be used. For dispersive clay soils or where the soil water chemistry leads to dispersion, the bridging of filter pores by tension cannot be relied upon.

Cracks or other potential openings allowing concentrated flow require special consideration. The problem is predominately with cohesive base soils, since non-cohesive soils cannot sustain an open crack when saturated. If a crack forms in a cohesive core upstream of a non-cohesive filter, the crack will extend into the filter until it collapses and self heals. Geotextiles do not necessarily possess this self-healing property unless it can extend and span the crack. As a general rule, therefore, geotextiles should not be used as a filter downstream of a cohesive core.

7.6.3 Differences Between Geotextile Filters and Granular Filters

Granular filters are normally used in embankment dams. Geotextiles may perform the same function, but there are differences in their nature and action that should be considered when replacing a granular filter with a geotextile.

Geotextiles often have better uniformity than granular filters. To ensure continuity, geotextiles should be overlapped about 2 feet. Geotextiles are subject to manufacturing tolerances under factory conditions, while granular filters are subject to the natural variability of soils and segregation during placement.

Geotextiles rely on their extensibility and strength to remain continuous during placement and subsequent deformation. They can tear or rip during placement or when subjected to concentrated displacement such as at a crack at the downstream face of a cohesive core. Non-cohesive granular materials cannot sustain a crack when saturated.

Geotextiles are much thinner than typical granular filters and are therefore inherently less conservative.

Filters are often used as intermediate transitions between fine soil and coarse soil. The transition acts as a filter to the fine soil and a base to the coarse soil. For granular filters, the self-filtering mechanism is the same at both interfaces. However, this is not necessarily true for geotextile transitions, where the movement of the fibers of the geotextile into the coarse soil is prevented by their tensile strength, similar to cohesive base soils. Success of the geotextile transition depends on the long-term continuity and tensile strength of the geotextile fibers. The bridging action of the geotextile has potential disadvantages. For example, geotextile laid beneath riprap on a cohesive base soil that has erosion gullies from rainfall, may span the gullies and allow runoff to continue to erode the base soil and perhaps generate fines that will clog the geotextile causing disruptive uplift pressures. Geotextiles used beneath riprap should be in close contact with the base soil. Covering with a rip rap bedding layer will help promote close contact.

7.6.4 Opening Sizes of Geotextiles

Different methods have been used to measure the effective opening size of a geotextile, leading to some discrepancies. A common method is to sieve graded sand through the geotextile. Variations include use of vibration, wet sieving, and dry sieving. Tiny glass balls are sometimes used instead of sand. Definitions of effective opening sizes based on the percent passing (or retained) also vary. The effective opening size resulting from the differing test methods and definitions can vary significantly and it is often prudent to perform tests with the specific soils and geotextile for critical situations.

7.6.5 Filter Design Criteria for Geotextiles

A variety of filter criteria has been proposed for geotextiles and are summarized in ICOLD (1986). After reviewing the criteria, the reference concluded that:

- The hydraulic conditions and critical nature of the interface should be carefully considered and the function of the interface defined.
- For non-cohesive base soils in one-way flow the geotextile should retain the d_{85} size of the base soil, which is less conservative than typical rules for granular soils involving the d_{85} size of the base soil. For well-graded base soils, a smaller geotextile opening may be required to prevent excessive particle movement before a stable interface develops. For uniform base soils, particular care should be taken, since a small error could result in all the soil particles being smaller than the geotextile opening size. Individual tests should be performed for gap graded soils and conservativism applied. Individual tests should be performed for all-important applications. For reversing or turbulent flow, more conservative criteria are required.
- For cohesive soils with non-dispersive conditions at the interface and without concentrated continuous flow through cracks, a fine-pored geotextile is satisfactory. If continuous flow can occur through a crack or other opening in a cohesive base soil, a granular filter should be used rather than a geotextile.

7.6.6 Permeability Requirements for Geotextiles

The transverse permeability required for filters is evaluated based on the allowable head losses in the filter-drain system, allowing for decreases in permeability from clogging and long-term deterioration. Granular filters are typically specified to be about 10 times as permeable as the base soil. The specified permeability of geotextiles should consider the potential for clogging and the reduction of permeability due to normal stress.

There is no apparent difference in principle between clogging of a geotextile and clogging of the superficial layer of a granular filter. However, clogging of geotextiles has received more attention, possible because it is much easier to observe. Several test methods have been used to evaluate the amount of clogging of a geotextile.

The permeability of a geotextile is less than indicated by the permeability of a clean sample of material due to the packing of fines at the face of the filter as well as blocking of pores within the filter. The rate and amount of clogging due to migration of fines depends on the type of base soil. Clays take longer to reach to reach a stable condition than do sands. ICOLD (1986) recommend that the transverse permeability be conservatively selected to be about 100 times the permeability of the base soil.

Other considerations regarding geotextile permeability:

- The permeabilities provided by manufacturers may be stated as flow per unit area at a given head or the permeability coefficient "k." If true permeability is to be obtained, the flow must remain laminar. Turbulent flow will result in lower permeability.

- The thinness of geotextiles results in small headloss across them. Therefore, for some applications the permeability of the geotextile may not be a critical parameter.

- Clogging of geotextiles beneath riprap has been observed to be more severe than for other applications. When subjected to uplift, this clogging combined with the tensile strength of the geotextile may lead to damage of the overlying riprap. In contrast, plugging of a granular filter would likely cause a local fracture that would relieve the pressure resulting in much less disturbance to the overlying riprap

- Longitudinal permeability and reduction of permeability as a function of normal stress

- Clogging by bacterial activity has occurred is some applications. Iron bacteria thrive in an iron rich environment with a pH between 6.0 and 7.6 and conductivity from +200 to +320 mV (EPRI 1992). Iron can be derived from natural soil deposits, rock, or from steel or iron features in the dam. Where iron bacteria are present, geotextiles probably should not be used as filters.

7.6.7 Geotextiles as Possible Shear Surfaces

A layer of geotextile within a soil mass forms a discontinuity that may represent a weakness in shear if the shear strength of the geotextile-soil interface is less than the shear strength of the adjacent soil. The extent to which the geotextile can form a weak plane depends on the extent to which it remains planar after installation. Tests of geotextiles in shear boxes show that the interface shear strength can be substantially less than for the adjacent soil, but depends on the type of geotextile and soil. In one series of shear box tests at low normal stresses, the interface shear strength was reduced 10 to 20 percent for loose sand and up to 40 percent in dense sand. Another series of shear box tests on undrained cohesive soil-geotextile interface showed that the effect of the geotextile was to destroy the pore water suction at low normal stresses resulting in much lower strength of the interface.

7.6.8 Consolidation and Seismic Activity

The compressive and tensile strains arising from initial construction and consolidation of an embankment dam are unlikely to exceed 5 percent and 2.5 percent, respectively. Impounding the reservoir may induce additional modest strains. Local strains can be much higher if compressible fills or foundations are involved. The compressive strain is of little consequence except as it may reduce the permeability of the geotextile. Geotextiles with moderate to high elongation at failure should be specified for use in embankment dams.

In areas of potentially high seismic activity, a geotextile with high elongation characteristics should be used. Strains from seismic activity may not be recoverable and accumulate over a series of seismic events. Use of geotextiles as the primary filter in areas where they may be subjected to seismic straining is questionable, particularly if cracking might occur that could induce severe local strains.

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CHAPTER 8 — REINFORCED ROCKFILL AND REINFORCED FILL

8.1 INTRODUCTION

Some material for this chapter is from ICOLD Bulletin 89, *Reinforced Rockfill and Reinforced Fill for Dams*, published in 1992.

Reinforcement of embankment dams is an old practice, probably beginning with using willow and rush mats to resist erosion during overtopping. Timber crib embankment dams are also historically old. Reinforcement allows embankment dams to resist tension loads and erosion forces, which they could not otherwise resist and has resulted in considerable cost savings when compared to more traditional alternatives.

The ICOLD Bulletin 89 provides an overview of the technology as it has been applied to dams. It presents numerous examples of how reinforced rockfill and reinforced fills have been used in dams, including diagrams and case histories. Aspects of the design and construction of reinforced rockfill and reinforced fill, specifically as it applies to embankment dams, is discussed. The reader is refereed to publications of the International Society of Soils Mechanics and Foundation Engineering (ISSFME) for discussion of the general theory of reinforced fill and to trade literature for discussion of the various proprietary systems and methods of design. Bulletin 89 is arranged in two parts; one for reinforced rockfill and the other for reinforced fill.

8.2 BACKGROUND

Reinforcement of earth materials is as old as recorded history and examples are common. Early military fortifications often consisted of embankments reinforced with sticks. Early roads were sometimes paved with logs laid crosswise (corduroy roads) so that carts and wagons would not get stuck in the mud. Other common examples of tension reinforcement and erosion protection used with earth materials are sand bags, timber cribbing, wire gabions, cellular steel sheetpiling structures, geofabrics, geogrids, soil nailing, and fiber reinforcement.

8.3 REINFORCED ROCKFILL AND REINFORCED FILL

Reinforced rockfill and reinforced fill used in dams generally have metal or plastic reinforcing buried in the fill and a downstream facing to retain the face of the fill. They are differentiated by the size of the fill particles and the nature of the facing required to retain the face of the fill.

Reinforced rockfill is rockfill that contains reinforcing, usually metal, and can be confined by a *discontinuous* facing.

Reinforced fill is fill that is finer than rockfill, contains metal or plastic reinforcing, and must be contained by a *continuous* facing.

In practice they are used for completely different purposes and have different appearances. Reinforced rockfill is used for protection from overtopping during floods. Up to now, it has most often been used as a temporary measure during construction to reduce the size and cost of diversion works, although it has been infrequently used for permanent spillways or overtopped weirs or dams. In appearance, it has a sloping downstream face that is retained by a steel mesh.

Reinforced fill is used to build entire dams, build overflow spillways, buttress dams, and raise dams. Up to now, it has been used on small to medium size embankment dams. In appearance, it has a vertical downstream face that is retained by reinforced concrete panels or narrowly spaced plastic grids.

8.3.1 Reinforced Rockfill

Rockfill is commonly used to construct embankments. Though its material properties vary, it generally has higher shear strength and higher hydraulic conductivity than other embankment materials. These properties often result in economical designs by allowing steeper slopes than can be used for other materials. Since rockfill has no inherent tensile strength, reinforcing can enhance its properties and result in additional cost savings.

Use of Reinforced Rockfill in Dams

Reinforced rockfill can be used in dam construction for:

- Diversion cofferdams that may be overtopped during construction
- Downstream face protection for embankment dams which may be overtopped during construction
- Steepening of slopes to reduce overall embankment volume
- Work platforms for heavy equipment and fill for construction roads
- Low dams or weirs subject to frequent and continued overtopping

Reinforced rockfill is used in other hydraulic works for:

- Protection of downstream works B channel lining, energy dissipaters, drop structures
- Throughflow or overflow rockfill weirs to control flood flows
- Reinforced mattresses to provide a stable base for an embankment or gate structure built underwater

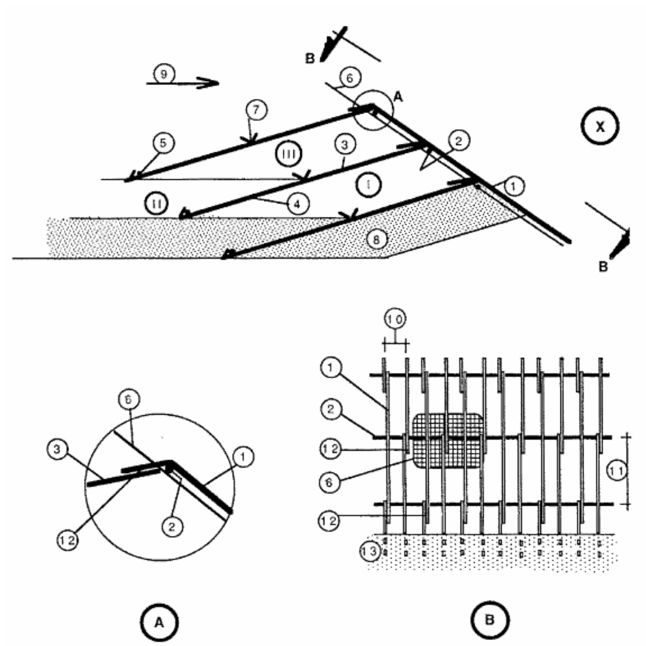
The first part of Bulletin 89 concentrates on the use of reinforced rockfill as downstream slope protection in dams and cofferdams. Its use allows the structures to be safely overtopped, should flooding occur during construction. Consequently, diversion works can be reduced (e.g., smaller diameter diversion tunnels) resulting in considerable cost savings.

Reinforced rockfill was first used to provide downstream face protection from overtopping in South Africa in 1917. Sheets of reinforced mesh were held in place by large rocks placed in a regular pattern. This simple system prevented serious damage when the dams were overtopped during construction.

More sophisticated methods for providing downstream face protection with reinforced rockfill have evolved over the last 40 years, primarily from research and construction experience in Australia, South Africa and the United States.

<u>Reinforced Rockfill Systems</u>

Two systems have been commonly used; the individual bar system and the gabion system. A typical individual bar system is composed of a surface mesh that is retained by a regular pattern of horizontal and vertical bars. The retaining bars are connected to horizontal anchor bars in the fill by tie bars that are approximately perpendicular to the face. Figure 8.1 illustrates the various components (from page 30, Bulletin 89) and Table 8.1 gives typical dimensions (from page 33; also shown below).



(X) Cross-section of a typical individual bar system. This system has no vertical dowel anchor bars. When used such dowels penetrate the previous layer to provide anchorage for tightening of anchor tie bars against mesh retaining bars before welding.

(A) Detail A.

(B) View B.

I
II Order of rockfill placement.
III

(1) Bar 1 (mesh retaining bar).

(2) Bar 2 (horizontal mesh retaining bar).

(3) Bar 3 (first anchor tie bar).

(4) Bar 4 (second anchor tie bar).

(5) Bar 5 (horizontal cross anchor).

(6) Surface mesh.

(7) Welded or mechanical splice.

(8) Previous layer.

(9) Flow.

Figure 8.1.

Table 8.1. Typical Dimensions of Reinforcement Used in Downstream Face Protection.

Item Description	Typical Dimensions	Remarks
SURFACE MESH		
Wire/Bar Diameter (mm)	Min.3 Avg. 5 Max. 20	Fencing mesh wire Reinforcing mesh wire Reinforcing bar
Opening Size (mm)	Min. 50 X 50 Avg. 100 X 100 Max. 1 200 X 300	Fencing mesh Reinforcing mesh Tied reinforcing bars
MESH RETAINING BARS		
Bar Diameter (mm)	Min. 20 Avg. 24 Max. 32	The down slope bars are normally installed on top of the horizontal bars
Spacing (mm)	Hor. Min. 300 Avg. 1 400 Max. 2 400	Generally depends on the size of reinforcing mesh sheets
Slope Distance (mm)	Ver. Min. 1 350 Avg. 1 500 Max. 1 800	Generally depends on lift, height and face slope dimension
ANCHOR TIE BARS		
Bar Diameter (mm)	Min. 20 Avg. 24 Max. 36	Approximately 200 to 350 mm^2/m; however, higher steel densities have been used
Length (m)	Min. 6.0 Avg. 12.0 Max. 40.0	Required length depends on stability criteria considered appropriate for design overtopping conditions
Spacing (mm)	Hor. Min. 250 Avg. 1 350 Max. 2 400 Ver. Min. 750 Avg. 1 000 Max. 3 000	Tie bar spacings and bar diameters depend on the size of the surface mesh sheets or gabion elements
ANCHORS		
Dowel Bar and Horizontal Cross Bar Diameter (mm)	Min. 20 Avg. 24 Max. 32	Dowels may be grouted into rockfill Horizontal cross bars are normally continuous between abutments and are generally tied to grouted dowels
Dowel Length (mm)	Avg. 1 000	

The primary advantage of the individual bar system is its lower cost relative to the gabion system. Its primary disadvantages are that assembly and connection must be done on the embankment, which can hinder rockfill placement.

In the prefabricated gabion system, modular gabion units are anchored into the fill by tie bars. Gabions consist of rockfill enclosed in wire mesh or geofabrics. The gabions may either be installed and filled on the embankment or be pre-filled and placed by crane. A system of gabions and their anchors can be designed and built for specific situations or proprietary gabion systems can be obtained for a wide range of applications.

The advantages of this system are the ease with which it can be installed and the minimal interference that it causes with fill placement. The primary disadvantage is its relatively higher cost.

Design of Reinforced Rockfill for Dams

The design of reinforced rockfill requires the evaluation of the service life, design conditions, surface mesh, and overall stability. Availability of hard durable rock of suitable gradation is crucial to the success of a reinforced rock fill system.

Service life

The service life of the reinforced rockfill must be established based on whether it will be a temporary feature (e.g., to protect the dam from overtopping during construction) or a permanent feature (e.g., to steepen the slopes, reduce the fill volume, or protect against overtopping). Corrosion of metal reinforcement is generally not considered in temporary reinforced rockfill, but is a primary consideration in permanent reinforced rockfill.

Design conditions

The depth, duration, and velocity of water flowing over the structure and the amount and velocity of water flowing through the structure must be established. Factors to consider include diversion procedures, construction rate, flood frequency, debris, aggressive water, flow concentration, and upstream and downstream discharge procedures.

Surface mesh

Surface mesh systems are designed empirically, because some loads, such as debris caught in the mesh, are not amenable to theoretical analysis. Knowledge of surface mesh protection systems which have survived water flowing through and over them currently provides the best basis for design. The mesh must retain the rockfill with water flowing through and over it. Generally, particles less than 50 mm are assumed to be washed out. The bar spacing must be compatible with the size of rock to be retained so that significant loss of rockfill does not occur. The rockfill adjacent to the mesh must be well compacted and interlocked to prevent individual particles from coming loose and abrading the surface mesh. Design of permanent surface mesh requires consideration of corrosion, and the effects of sediment load in the water flowing through and over reinforced rockfill. Chain link wire fencing has been used for surface mesh. Failure of at least at one dam has occurred and chain link should be used with caution.

Overall stability

Slip circle or sliding wedge analyses should be done to determine the depth to which reinforcement should be extended into the rockfill. A variety of methods are available including design charts for some proprietary systems and conventional slope stability analyses. Computer programs which model embedded reinforcement are available.

The effect of both buoyancy and seepage forces must be evaluated. Toward the bottom of the slope, the combined forces tend to wash rockfill particles out the slope. If not retained by the mesh, the surface flow will wash them away.

Construction Considerations

The following items should be considered in order for reinforced rockfill to provide a high level of downstream face stability during all phases of construction.

- Installation has to proceed in unison with the fill placement.
- When overtopping is imminent, the downstream face profile should be weir shaped with a uniform crest height to prevent flow concentrations.
- The connection between the reinforced fill and the abutments must be protected from erosion.

Bulletin 89 gives examples of design and construction of reinforced rockfill for Clarrie Hall Dam (Australia), Googong Dam (Australia), Fika Patso Dam (South Africa), Murchison Dam (Australia), and Crotty Dam (Australia). The design of Pit 7 Afterbay dam (California) is described in Shackelford, et al. (1970). The design of the Henshaw Dam Modifications is described in Bischoff et al. (1985).

Performance

Where reinforced rockfill has been subjected to flowing water, it has generally had satisfactory performance, although some failures and partial failures have occurred.

The depth of overtopping which reinforced rockfill can safely withstand has not been clearly established. As a rule of thumb, depths in excess of 3 m may be considered to be the upper limit, although overflow depths up to 10.5 m have been sustained without damage. Permanent deformation of downstream faces has occurred from overflow, though it generally has not affected the stability of the reinforced rockfill.

Of the 47 known cases where reinforced rockfill has been used to protect main dam embankments from overtopping, 21 have been overtopped, and only 4 have failed.

Failures have generally been progressive or partial failures that occurred as the result of key portions of the reinforced rockfill system being damaged during significant overtopping. References are supplied in ICOLD Bulletin 89 for the dams that have been overtopped.

8.3.2 Reinforced Fill

Reinforcing provides the fill with tensile strength, which it otherwise would not have. Consequently, fills with steep or vertical faces can be constructed, resulting in considerable cost savings.

Reinforced fill is comprised of three basic components; fill, reinforcing, and facing. The fill is generally relatively clean granular earth material. The reinforcement is usually metal, geofabric, or geogrid laid in horizontal beds. The facing is usually precast concrete elements, metal, or geofabric. It is connected to the reinforcing and retains the face of the fill.

Use of Reinforced Fill in Dams

Reinforced fill can be used in dam construction to:

- Construct entire dams
- Build spillways
- Buttress dams
- Raise dams

Reinforced fill can be used in works associated with dam construction for:

- Access supports
- Spillway channels

The second part of Bulletin 89 concentrates on the use of reinforced fill for construction of dams. Use of reinforced fill has been limited to small to moderate size dams. It is generally used as a downstream shoulder with a vertical downstream face. The upstream shoulder and impervious element are the same as for a conventional embankment dam. The impervious element may be either a thin layer on the upstream slope, or a thick impervious fill that forms a core or upstream shoulder. A drain is usually placed between the impervious element and the reinforced fill. Reinforced fill has been used to raise dams with the fill comprising the entire raise. However, this case is not covered in the report.

Reinforced fill is particularly attractive for use in construction of an overflow spillway section of a dam. The vertical downstream face reduces the volume of the embankment and provides for a free-falling nappe. The top of the reinforced earth body must be protected from the overflowing water by a concrete apron and an energy dissipator is required downstream of the fill.

Reinforced Fill Systems

Reinforced fill in dams may use a proprietary method such the Reinforced Earth system or the Websol system, the ladder wall method, or a unique design.

Reinforced Earth

A system called Reinforced Earth, that has been widely used in civil engineering projects and in a few dams, is a proprietary system developed by Mr. Henri Vidal, a French engineer and architect. Reinforced Earth typically consists of backfill with horizontal beds of parallel reinforcing and a facing. Figure 2 shows a typical Reinforced Earth fill (From Bulletin 89, page 68).

The reinforcing is typically spaced several tenths of a meter vertically and about one meter horizontally. The reinforcing is commonly galvanized flat steel strips that are 4 to 10 centimeters wide and several millimeters thick. They are often ribbed to increase frictional resistance to pull out.

The most common facing is precast concrete panels, typically 0.1 to 0.2 m thick, with an area of about 2 m^2. The reinforcement is bolted to the panels. Semi-circular metal shells laid horizontally have been also been used.

The fill is commonly limited to clean granular material because it has a high coefficient of friction with the reinforcement. In practice, material with more than 10 percent passing 20 microns is unsuitable. Clay has been used as fill for some Reinforced Earth structures, but not dams. Clay has potential problems, such as swell, creep, and low coefficient of friction, and its use in reinforced fill is not discussed in the bulletin.

The Reinforced Earth system was first used at a dam, the Vallon des Bimes Dam (France), in 1972. Since then, a dozen small to moderate size dams or portions of them have been constructed using reinforced fill. This system is used as the primary example throughout the bulletin, though some information is given on the other systems.

Websol System

The Websol system is a proprietary system, similar to the Reinforced Earth system. The primary difference is that it uses of polyethylene coated, polyester fiber, multicord anchors to retain the facing. The Websol system was recently used to raise Googong Dam (Australia).

Ladder Walls

The ladder wall method was developed by André Coyne in the 1930s and was first applied to a dam in 1940. In this method, the upstream face is composed of reinforced concrete wall that is vertical or slightly inclined and forms the impervious element. The wall is held in place by steel rods connected to anchor plates embedded in the fill. The primary difference is that the reinforcement relies on passive pressure developed by anchors, rather than friction. The ladder wall method was used to build the Laurenti Dam (France) and the Saint-Cassien Dam (France). The ladder wall was used for both upstream and downstream faces at Conqueyrac Dam (France).

Generic Designs

Generic reinforced fill uses the same principles and techniques as the other systems; however, the individual components are designed and built for the specific application. For example, geofabric was used to reinforce the fill at Maraval Dam (France).

Design of Reinforced Fill for Dams

The bulletin concentrates on the design of overflow spillways constructed with a downstream shell of reinforced fill. The structure consists of two relatively independent parts with regard to their design. The upstream shoulder is designed by conventional embankment dam methods. The downstream shell of reinforced earth is designed as discussed below.

When a proprietary system is used, the design of the reinforced fill is usually done by the supplier. Whether, or not, the design is done by the supplier, the engineer responsible for the project must be satisfied with the design. The design of reinforced fill is discussed to enable the responsible engineer to evaluate the design of a reinforced fill body and its elements. It is based on the French standard NF P 94-220 (July 20, 1992) for metal reinforcement; but the problems and solutions would be similar for other materials.

Durability and Design

Reinforced fill may have a limited service life if metal reinforcement is used because of the gradual loss of strength of the reinforcement due to corrosion. At the end of the service life, the stresses in the reinforcement must still be less than or equal to allowable stresses. Since experience with reinforced fill is limited to about 30 years, precautions must be taken against corrosion to obtain adequate durability.

A specific study must be undertaken including:

- Evaluation of the corrosive nature of the environment to which the reinforcement will be exposed, specifically, the water and fill quality
- Evaluation of the reinforcement with respect to corrosion and other effects of aging
- Evaluation of the monitoring necessary to verify the actual rate of corrosion
- Study of the methods by which the dam will be made permanently safe at the end of its service life

For a Reinforced Earth dam with a 100-year service life in a non-aggressive environment, the reinforcement would be made of galvanized steel with an extra 2 mm of thickness for protection from corrosion. Vinyl coating could also be used to resist corrosion of metal reinforcement.

Preliminary Sizing

For most applications, the reinforced fill body has a rectangular cross section and the economic length of the reinforcement is roughly 70 percent of the height of the wall. Figure 3 shows a typical reinforced fill used in an overflow spillway (from page 92). In cams, the length of the upper reinforcement is sometimes reduced to limit the length of the spillway slab and the overall amount of fill.

The depth of embedment in the foundation usually depends on the expected amount of scour. For reinforced fill not subject to overflow, the depth of embedment is typically 5 percent of the wall height, though would be less for competent rock foundations and more for poor soil foundations.

Calculation of Internal Stresses

The reinforcement must be strong enough so it will not fail in tension and must be long and wide enough so that it will not pull out of the fill. The facing must resist the active earth pressure acting on it.

The phreatic surface is determined as would be done for a conventional embankment dam. The effective stress and seepage pressures must be included in the stress calculations.

The method in the French code for calculating internal stresses concisely accounts for the main physical laws of reinforced fill behavior and has been checked by numerous laboratory and field tests. More detailed theoretical studies have been done and may be found in the references.

The reinforcement restrains, through tension, an active soil wedge adjacent to the downstream face. The maximum tensile stress occurs approximately at the line between the active wedge and the remaining fill. It is calculated using an experimentally determined earth pressure coefficient and the contributing area of the fill. Equations for the maximum tensile stress and experimentally derived earth pressure coefficient are included in the bulletin.

Overall Reinforced Fill Structure Stability

The overall stability is verified by checking the bearing capacity of the foundation, the shear resistance at the base of the reinforced fill, slope stability, and settlement. Internal strength of the fill is critical to the design.

Bearing capacity and sliding along the base are checked by conventional methods, treating the reinforced fill body as a single unit with a width equal to the length of the reinforcement.

The stability of potential failure surfaces through the fill are evaluated with conventional slope stability methods, taking into account the reinforcing intercepted by the failure surface.

Total and differential settlement is calculated in the conventional manner. The facing and adjacent structures must be able to tolerate the estimated settlements.

Load Cases

Overall stability and internal stresses should be evaluated for the following cases.

- Normal water levels
- Floor water levels
- Overtopping during construction
- End of construction
- Accidental saturation
- Earthquake loading

The primary difference in the first five cases is the location of the phreatic surface within the embankment. The phreatic surface is determined as would be done for conventional embankment dams for normal and flood water levels.

Overtopping during construction might result in saturation of the upstream shell and the reinforced fill. Stability must be maintained, although with reduced factors of safety. The entire thickness of the reinforcing (including corrosion allowance) should be used.

Pore pressures from placement and compaction of the fill is accounted for in the end of construction case. Generally, pore pressures do not occur in reinforced fill because of the requirement that the fill be free-draining.

The combination of clogged drains and an extreme flood occurring near the end of the service life could result in saturation of the entire embankment. Though this is an unlikely case, sometimes it can be withstood by reinforced fill with little or no extra cost. In this case, the reinforcement is assumed to have lost the extra thickness provided for corrosion.

Earthquake loads must consider the response of the reinforced fill and the possibility of overstressing and failing the reinforcement within the fill.

Construction Considerations

The construction of reinforced fill needs to be carefully planned so that it does not restrict the rate of placement of the rest of the fill. First the foundation is excavated, a leveling pad for the facing is constructed, and the first layer of facing is placed. Construction proceeds by alternating the following operations: spreading and compaction of a fill layer, placement of a row or reinforcement, placing a layer of fill, and placing the facing.

Specifications, placement, compaction, and erection criteria, and quality assurance requirements for the various reinforced fill components, with specific examples for "Reinforced Earth" are provided in Bulletin 89.

Performance

Reinforced fill dams are considered to perform adequately when there is little residual settlement, limited outward leaning of the downstream face, controlled seepage, and minimal erosion. Few performance anomalies have been observed in reinforced fill dams, so the discussion is limited to hypothetical examples of what would constitute, inadequate performance and how it would be remedied. Excessive deformation, abnormal internal seepage, cracking of the spillway slab, deterioration of the facing, corrosion of reinforcement, and downstream scour are discussed in Bulletin 89.

Settlement, deformation, seepage, and erosion are monitored with visual observation and conventional instrumentation. Reinforcement corrosion is monitored by burying samples in the fill during construction and periodically removing and testing them.

8.4 APPENDICES

The appendices in Bulletin 89 provide a list of dams with reinforced rockfill, figures and discussions of four dams with reinforced fill B The Vallon des Bimes Dam (France), Taylor Draw Dam (USA), Googong Dam (Australia), and Conqueyrac Dam (France) and calculation details for internal stresses in reinforced fill.

8.5 REFERENCES

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CHAPTER 9 — UPSTREAM SLOPE PROTECTION MATERIALS

9.1 INTRODUCTION

The upstream slopes of embankment dams are exposed to dynamic and mechanical attack by wave action and climatic weather conditions. The materials used to prevent damage must resist these forces and conditions. Chapter 8 does not deal with the design of the slope protection, but describes the type of materials commonly used and their physical properties. Chapter 8 also does not consider materials for the facings for rockfill dams which are discussed in Chapters 5 and 6. ICOLD Bulletin 91, *Embankment Dams Upstream Slope Protection*, with its numerous references, is a source of detailed design procedures and use of materials. The majority of the material for this section is taken from ICOLD Bulletin 91 (2).

The following types of upstream slope protection materials are covered in this chapter.

- Dumped stone riprap
- Hand placed riprap
- Soil-cement
- Roller compacted concrete facing
- Concrete paving and precast concrete blocks
- Bituminous concrete lining
- Gabions and Reno-Mattresses
- Steel and timber facings

The bedding layer required for most slope protection materials is very important to the performance of the upstream slope protection. The materials for the bedding layer are discussed briefly in this chapter and Chapter 4, as well as ICOLD Bulletin 95, Embankment Dams - Granular Filters and Drains.

9.2 DUMPED STONE RIPRAP

9.2.1 General

The main purpose of riprap on embankment dams is to prevent erosion and damage from wave action. Rock fragment dumped riprap is the most common type of slope protection used for embankment dams. Quarried rock is the most common source of rock fragments. The exploration for sources and the quality evaluation is similar to that used for rockfill materials described in Chapter 3. The main difference is that the requirements for gradation and durability are much more stringent.

Riprap must contain a high proportion of near maximum size fragments required by design considerations to resist wave attack and should contain enough smaller rock fragments to fill the voids and lock the larger stone in place. The riprap should be composed of dense, sound, durable rock fragments with near cubical shape as possible. Specification for construction frequently requires the ratio of the maximum to minimum dimension of the rock fragments shall not exceed 3. This is important because the higher the unit weight, or relative density of the placed riprap, the better it will be able to resist wave damage. It should be placed without segregation, and in as dense and interlocked state as possible. This requires machine or hand manipulation of individual rock fragments. The provision of a bedding layer, or layers, is essential to the successful performance of the riprap. Only in rare instances can a bedding be eliminated where the underlaying material meets filter criteria or the riprap layer is very thick and little wave action energy is left to erode the underlying embankment.

9.2.2 Quality of Rock for Riprap

Quality evaluation of riprap relies on petrographic examinations, in addition to examination by experienced and qualified personnel supplemented with data from laboratory durability testing. Petrographic examinations may reveal defects in rock which may seem satisfactory from laboratory testing. Because of the large size of the rock fragments required, the testing needs to be performed on small representative samples. The laboratory tests for durability commonly performed are listed in Table 2, in Chapter 3. Ideally rock for riprap should meet the quality specifications of concrete aggregate. No minimum quantitative specifications can be given for the rock quality; the best available material should be used. Generally natural boulders, because of their more rounded shape, will have poorer interlocking than quarried rock pieces and slightly lower resistance to wave action for the same piece size. Elongated or flat pieces will have less stability for the same mass than equi-dimensional pieces when dumped randomly (5).

9.2.3 Production of Riprap

The production of riprap using only hand labor for the selection, sizing, and construction has not been considered practical since the early 20th century. Methods using shovels, backhoes, and loaders for selecting, sizing, and loading are still used today. However, it is becoming much more common to process the rock for riprap by scalping, crushing, and screening in a plant.

Quarry selection is probably the most important item in producing acceptable quality riprap. The best available source considering economics should be selected. Blasting uses a wide variety of drill hole patterns and explosive factors. The density of the drill hole pattern and powder factor used is determined by the geological conditions in each quarry and the maximum rock fragment size requirement. It is much more common today to optimize the hole density and powder factors by utilizing computer programs to model a blast. The programs estimate fragmentation and gradation for various combinations of hole patterns and powder factors. However, past performance, field adjustment and experience are important to optimum and satisfactory production.

9.2.4 Field Placement and Control

Placement methods have evolved with the advent of new equipment. The first consideration in placement is that the quarry operations are such that a good mixture of rock sizes is available in each load delivered to the site. Gradation tests are occasionally performed on loads before placement to ensure that size distribution requirements are met. Placing of the loads should be performed to ensure that segregation does not occur. Placement is accomplished by placing loads along the slope against previously placed riprap to prevent the segregation that occurs if dumped in piles. Dumping from the top of a slope into a chute is not allowed. Dumping should proceed in horizontal rows and progress up slope. In recent years backhoes, and Grade Alls, with 1.1 to 1.9 m^3 (1.5 to 2.5 yd^3) capacity buckets have become the most common method for placement. With backhoes, and Grade Alls, the riprap must be kept close to embankment level for the arm to reach below the slope. Other successful methods for placing riprap include dragline with skip, cranes with clamshells, and rubber tired front end loaders. Continual visual inspection is required during placement to ensure proper mixing and interlocking of the rock fragments. In most cases some reworking by hand is required, but it can be minimized by proper loading and placement (5).

9.3 HAND-PLACED RIPRAP

The quality of hand-placed riprap is similar to that required for rockfill or dumped riprap material. The main difference is particle shape requirements and fragment size. Hand-placed riprap consists of stones carefully laid by hand in a single layer in a more or less definite pattern with a minimum amount of voids and with top surface relatively smooth. Rounded or irregular rocks lay up less satisfactorily and less rapidly than rock that is roughly square. The flat, stratified rocks should be placed with their large axes aligned up and down the slope. Joints between large rock fragments should be offset as much as possible, and joint openings to the underlying fill should be avoided by carefully arranging the various sizes of fragments and closing the openings with spools or small rock fragments. However, there should be enough openings in the surface of the riprap to allow the water pressure to dissipate without lifting the rocks (6).

Hand-placed riprap is satisfactory when not exposed to heavy ice conditions. The rock must be of better quality than the minimum suitable for dumped riprap. It should be recognized that hand-placed riprap is not as flexible as dumped riprap, because it does not adjust as well to foundation or local settlements. Consequently, hand-placed riprap should not be used where considerable settlement is expected.

Some years ago, it was generally believed that a layer of hand-placed riprap offered the same protection as a layer of dumped riprap of twice its thickness. Experience has shown, however, that hand-placed riprap is no more effective than dumped riprap of equal thickness, and perhaps even less effective (9). The single hand-placed layer is vulnerable to the displacement or disintegration of individual stones, and being more rigid than dumped riprap it is less able to adjust to local movements or settlements of the embankment. Because of the increased costs of labor and equipment and the lack of any advantages over dumped riprap, except perhaps with respect to the appearance of the protected slope, hand-placed riprap is now rarely used (5).

9.4 SOIL-CEMENT SLOPE PROTECTION

Soil-cement as slope protection material for embankment dams has been found to be economical where suitable riprap is not available near the dam site. The soil-cement is generally placed and compacted in horizontal layers about 6 inches thick. The thickness of the soil-cement slope protection is between 2-4 feet normal to the slope. This is usually determined by the practical minimum placing horizontal width of 8-12 feet.

Soil-cement is a mixture of soil, portland cement, and water. Through compaction and cement hydration, the mixture hardens forming a dense, durable, relatively impermeable, erosion resistant material. The soil-cement can be made with a wide variety of mineral silty soil (SM-SC). The main criterion is gradation. Soils with more than 15-20 percent minus 200 sieve size or very cohesive are not suitable or economical because of the large amount of cement needed and difficulty in production.

Standard laboratory tests are used to design and verify the proportions and quality of the soil-cement slope protection.

9.5 ROLLER COMPACTED CONCRETE SLOPE PROTECTION

Roller compacted concrete is very similar to soil-cement slope protection. The most significant difference is the gradation and quality of the aggregates. The aggregates for roller-compacted concrete are more like concrete aggregates than soil. The resulting material is very similar to soil-cement. Roller compacted concrete is frequently used for downstream slope protection for small dams when additional spillway capacity is needed to pass rare floods.

9.6 CONCRETE PAVING AND PRECAST CONCRETE BLOCKS

Concrete paving deserves serious consideration for upstream protection where riprap is too expensive. Concrete paving is used on both earthfill and rockfill dams, although its performance on rockfill dams has been much better, especially on well compacted rockfill dams. The success of concrete paving as a slope protection medium depends on the field conditions, on the behavior of the embankment, and on the ability of the paving to resist cracking and deterioration. Concrete pavement has proved satisfactory in some cases under moderate wave action. Where severe wave action is anticipated, concrete pavement appears practicable only when the settlement within the embankment after construction will be insignificant (6).

If a complete history were gathered concerning the numerous instances where concrete paving was used for the protection upstream slopes of small dams, the number of failures would be tremendous (about 36 percent). Unfortunately, the fact that some structures protected with concrete paving have withstood the test of time, continues to lead engineers to use this type of construction often without sufficient reference to other unsatisfactory performance records. A properly designed and constructed concrete paving is never cheap. The uncertainty and complexity of the forces that may act on a concrete paving make conservative treatment desirable whenever this type of slope protection is considered (5 and 6).

The materials for slope protection concrete must meet standards and criteria for structural concrete in respect to strength and durability. Refer to Chapters 3, 4, and 6 for technology for the production and quality control for concrete aggregates.

9.7 BITUMINOUS CONCRETE LINING

Bituminous linings have been economically and successfully used for the dual purpose of providing an impervious membrane as well as upstream slope protection. Dutch experiences in asphalt slope protection works could also be introduced to high dams and reservoirs (10).

Refer to Chapter 5 for the use of asphalt concrete as the water barrier in embankment dams. Chapter 5 includes the technology for the production and quality control for bituminous concrete asphalt materials.

9.8 GABIONS AND RENO-MATTRESSES

Gabion mattresses are rectangular wire boxes filled with rock. The mattresses are wired shut after filling and stacked on top on one another to form a stepped slope. The wires are often galvanized, or PVC-coated to provide some corrosion resistance. This method is generally excellent for steep side slopes.

'Reno-mattresses' are similar to Gabion mattresses, except that these mattresses have less height and are laid end to end up the incline, instead of being stacked upon each other (11).

The wire cages are built in large sections, instead of individual boxes, and are kept in place by the friction between the mattresses and the embankment. No footings are necessary; however, protection at the toe will be required to prevent scour (5).

Gabion and Reno-Mattresses are generally used for small structures where large rock fragments are not available, or are not economical. The wire boxes serve to increase the effective size of the rock fragments. The maximum size of rock fragments is generally 2 to 5 centimeters. The production quality of the rock fragments is the same as for rock riprap discussed previously.

9.9 STEEL AND TIMBER FACING

Both steel and timber facings for rockfill and gravel dams have been used successfully on a number of dams to form the impervious membrane, and incidentally, the upstream slope protection. Some of these steel facings are more than 70 years old and are reported to have required little maintenance (5).

Materials for steel and timber facings are obtained from commercial sources and are not discussed in this Bulletin.

9.10 REFERENCES

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CHAPTER 10 — MATERIALS FOR WATERTIGHT CUTOFFS

10.1 INTRODUCTION

Cutoff walls have been used extensively to rehabilitate existing dams that have been found to have deficiencies. They have also been used in the design of some new dams. In all cases, the objective was to provide a low permeability element (or diaphragm) within the embankment and/or its foundation either as the primary seepage barrier or to supplement existing barriers.

Cutoff walls may be classified according to their stiffness, the type of backfill materials used, or construction methods used. The intent of this chapter is to provide a brief description of various types of cutoff walls, the materials used in their construction, and some information on their construction as it applies to the material selection.

10.2 TYPES OF CUTOFFS

The term diaphragm is a general term used to distinguish a water retention element that is thin compared to the surrounding embankment. Diaphragm walls are typically located near the centerline of the dam. This places the wall in the part of the dam that has nearly balanced stresses and thus should produce the least affect on the embankment=s stability and the least movements after wall construction. The top of the wall can either be on or near the crest if cutoff of the embankment itself is necessary or at the base of the embankment if cutoff of only the foundation is necessary.

Sometimes a wall may be constructed from the upstream face of the embankment or even at the upstream toe. In such cases, the stability of the embankment due to the strength characteristics of the slurry filled trench and eventual backfill must be considered. Such stability considerations may affect the length of trench that is allowed to be open at one time or even the type of construction allowed. For instance, panelized construction techniques may be selected over long open trench excavations to allow for the embankment stresses to be spread to adjacent panels.

10.2.1 Wall Construction Techniques

Selection of the material to be used in wall construction is dependant on the construction technique to be used to build the wall. Some backfill materials are better suited to certain wall construction techniques than others. Options for wall construction are dependant on such things as the difficulty or ease of the material to be excavated, accessibility of the site, reasonable availability of equipment, and desired properties of the wall (permeability, deformability, crack stopping ability, etc.).

Panelized Construction

A panelized wall is one in which a series of primary and secondary panels are excavated and backfilled with material in-situ. Panelized construction is well suited to sites where difficult, slow excavation is expected. In specialized cases, cages of steel reinforcement can be inserted in the panels prior to backfilling to give additional strength to the wall.

Backfill materials well suited to panelized construction include: structural concrete, plastic concrete, and soil-cement-bentonite.

Typical construction equipment includes mechanical or hydraulic clam shells or hydromill excavators. Selection of equipment type is based on the difficulty of the material to be excavated and the contractors familiarity with the equipment. In some cases, a continuous set of large diameter drill holes drilled by reverse circulation methods have been used.

The construction of a panelized wall generally involves the following steps:

1. The excavation of a diaphragm is usually preceded by the construction of shallow concrete guide walls that are used to align the excavating equipment, add stability to the top of trench, and aid in quality control. These walls are typically left in place following construction completion.

2. The excavation begins with the excavation of a primary panel typically equal to a minimum of three >bites= of the excavating equipment. The two outside bites are excavated first followed by the remaining middle bite. The excavation is kept open by use of a bentonitic slurry. Once excavated, the bottom of the primary panel may be cleaned, the solids in the slurry reduced, and the panel backfilled with the selected material. Backfilling is usually accomplished by backfilling tremie pipes from the bottom up keeping the pipe bottom below the top of the backfill.

3. Once two of the primary panels have been backfilled and allowed to attain some set of the backfill, the intervening panel of foundation material is removed and the secondary panel is constructed. Large diameter stop-end tubes may be used at the ends of the primary panels to provide a better joint with the secondary panel. More recently the dimension of the secondary panel is left smaller than the excavating equipment such that during excavation of the secondary panel, equipment trims some of the primary panels side to provide a clean, fresh joint.

4. Backfilling of the secondary panel completes the construction.

<u>Continuous Trench Construction</u>

Walls constructed by the continuous trench method involve the excavation of the foundation material in one continuous operation. The method is best suited to foundation material that can be rapidly excavated, but slower construction techniques can be employed with some types of backfill material. In some cases, continuous trench excavation can be used for the upper part of the trench followed by panelized construction below to obtain final depth.

Backfill materials well suited to this type of construction include: soil-bentonite and cement-bentonite.

If soil-bentonite is used, the excavation is kept open with a bentonite slurry until a considerable amount of trench has been opened. Backfill is then pushed into the trench from one end, maintaining a slope of about 10H:1V to efficiently displace the slurry. Complete displacement of the slurry relies upon the differential density between the soil-bentonite backfill and slurry. Soil-bentonite walls often require a significant trench width (1.5 to 3m; 5 to 10 ft) to ensure adequate head loss across the wall and to accommodate typical construction equipment.

If cement-bentonite is used, the material is used as trench support from the very start of excavation and is left in place as the final product. Shutdowns are accommodated by the re-excavation of the set material (which is wasted) followed by the continuation of the excavation process. Foundation material must be capable of rapid excavation since the cement-bentonite begins to set as soon as it is introduced - although retarders can be used to delay the set. Cement-bentonite material is not conducive to placement by displacement of a bentonite slurry (as for soil-bentonite) due the lack of sufficient differential density between the two materials. Cement-bentonite is not used in wall applications requiring the use of a hydromill excavator since the cement will typically cause plugging of the equipment.

Shallow concrete guide walls may or may not be used to help align excavating equipment and to add stability to the foundation material at the top of the wall.

Other Techniques

In addition to the conventional diaphragm walls and slurry trenches described above, deep watertight cutoffs also can be constructed using technologies such as inplace soil mixing, bored-pile systems, injected grout screens (curtains), prefabricated interlocking concrete walls, and interlocking geomembrane systems. When a shallow watertight cutoff is required, a compacted backfill trench is often the most economical choice. These systems are briefly described.

In-Place Soil Mixing

In-place soil mixing is a technology that was originally developed in Japan during the late 1960's and early 1970's. It consists of mixing in situ soils with cement grout using multiaxis augers and mixing paddles to form a row of overlapping soil-cement columns. The installation is feasible in many ground conditions, including gravelly and cobbley soils, and soft rock. Details of the technology, equipment, and end-product characteristics are summarized in Xanthakos (1994) and Taki and Yang (1989).

Bored Pile Systems

Secant piles or interlocking piles were among the first technologies used to construct deep cutoffs. Bored pile walls can be contiguous with the piles in contact or built as an interlocking wall with overlapping elements also called a secant pile wall. Joints must be sealed by grouting. In general, bored pile systems are not as watertight as diaphragm walls. These systems are often associated with cofferdam construction. Examples of this application are given in Alfonso, et al. (1984) and Bruce and Stefani, (1996)

Injected Grout Screens (Curtains)

Originally, thin screens or curtains of bentonite-cement were constructed by driving a group of H piles into the foundation then extracting them one at a time to form a continuous slot in the ground. A thin, impervious screen was formed by injecting clay-cement grout beneath each pile as it was being extracted. This construction process has been essentially replaced by the vibrating beam technique.

The vibratory driver/extractor is a machine that both drives and extracts a pile without changing equipment. This device permits reuse of the same wide flange beam which is repeatedly inserted and extracted, with grout injected through the bottom of the beam during extraction to form overlapping elements of a continuous wall. The process is much more efficient and less costly than the original method (Leonards, et al., 1985).

Prefabricated Concrete Diaphragm or Interlocking Geomembrane Walls

Prefabricated concrete or interlocking geomembrane panels are installed in slurry-filled trenches using a similar construction sequence as outlined for conventional diaphragm walls. Instead of the in situ tremie placement of concrete, the precast concrete or geomembrane panels are inserted with the aid of guide walls (Bliss, 1995). Single grout or displacement grout must be used with the precast concrete elements to ensure that an effective seal is created between panels. The geomembrane panel joints are typically self-sealing with patented interlocking features. In some soil conditions, the geomembrane panels can be installed using a vibratory hammer and insertion plate eliminating the need to excavate a slurry trench.

Compacted Soil Cutoff Trench

A compacted soil cutoff trench provides an effective impermeable barrier when the pervious deposits in the foundation can be completely penetrated by conventional excavation. The cutoff trench is an extension of the impervious core zone of the embankment and must usually be excavated to an impervious foundation material to ensure continuity. Placement and compaction control of trench materials should be the same as for the core, with special care taken at the bedrock/core contact. As a general rule, the base width of the trench should be about 1/4 of the maximum head difference between reservoir pool and tailwater.

10.3 MATERIALS

10.3.1 Slurry Materials

Important Properties

For types of walls where temporary slurry is used, the slurry performs many critical functions including (Xanthkos, 1979):

- Support the face of the excavation and also prevent the soil from sloughing and peeling off;
- Seal the formation and form the filter cake preventing slurry loss to the ground;
- Suspend detritus, thereby preventing sludgy unconsolidated layers from accumulating at the bottom of the trench;
- Carry the cuttings in the slurry volume, thereby preventing sedimentation in the mud circuit;
- Ensure free flow of backfill from tremie pipes to allow complete displacement by fresh backfill without affecting the development of bond;
- Flow in pipes to facilitate materials handling from the excavation;
- Aid sedimentation in tanks and permit the separation of solids in shaker screens or cyclones; and
- Facilitate their own disposal in dump areas or in public drains.

The properties of slurry that may be specified, depending on the application, include: viscosity; density, unit weight, or specific gravity; filtrate loss; and pH. These properties are typically controlled not only for the freshly prepared slurry prior to installation in the trench, but also for the slurry while in the trench.

Viscosity

Slurry viscosity is measured with a Marsh funnel (American Petroleum Institute Specification 13A). Slurry viscosity must be low enough that it is easily displaced by backfill or tremie concrete, but high enough to maintain suspension and trench stability. A typical bentonite slurry viscosity of about 40 Marsh seconds satisfies these requirements. Fresh slurry should have a minimum viscosity of 32 Marsh seconds, and slurry in the trench typically should not exceed 65 Marsh second viscosity (Millet, et al., 1992).

Density

Fresh slurry density is typically maintained slightly above that of water, about 1.04 to 1.15 g/cm^3 (65 to 72 lb/ft^3). Density is measure with a Mud Balance test (American Petroleum Institute Specification 13 B-1, Section 1). In the trench, slurry density may increase to 1.25 g/cm^3 (80 lb/ft^3) as it acquires suspended fines and sand particles. A general rule is that the unit weight of slurry in the trench should be at least 15 lb/ft^3 (0.24 g/cm^{-3}) lower than the unit weight of the backfill material (D=Appolonia, 1980).

Filtrate Loss

Filtrate loss relates to the formation of a Afilter cake@ as the slurry penetrates the porous medium along the trench walls. The formation of the filter cake is crucial to maintain stability of trenches in pervious granular materials. The nature of the filter cake depends on the degree of peptization or flocculation of the suspension. As a stable slurry is filtered at the trench wall, a thin, impervious and compact filter cake is formed. A flocculated suspension will deposit a relatively porous and thicker layer that can have much higher permeability. Factors that cause flocculation include high salinity of the groundwater, low pH, and high concentrations of Ca^2+ ions (the latter caused by addition or excessive contamination by cement).

Filtrate loss and corresponding cake thickness are simulated by a filter press test (American Petroleum Institute Test PP131B). The normal range of filtrate loss for bentonite slurries is 15 to 30 cm^3. Filtrate loss for cemented-bentonite slurry can be significantly higher, ranging from 100 to 180 cm^3. In general, the bentonite slurry should be fully hydrated before cement is added to preclude high filtrate losses. High filtrate losses (in excess of 30 cm^3 as measured by the API procedure) indicate the trench is in danger of collapse (Millet, et al., 1992).

pH

pH is an important property of the slurry if soil or groundwater conditions could significantly alter the pH of the fresh slurry. The pH ideally should be maintained in the range of 6.5 to 10. The slurry should be carefully monitored if the pH exceeds 10.5.

Materials used to make trench-stabilizing slurries include clays, cements, admixtures, and bio-polymers. These materials are briefly described in the following sections.

Clays

Sodium bentonite clays are the most common materials used to make trench-stabilizing slurries. The important properties of bentonite relevant to slurry performances are: swelling, viscosity of suspensions, liquidity, and plasticity. These properties are variable in natural bentonites. If a premium grade bentonite is used with good quality mixing water, the slurry will require only about 5 to 7 percent bentonite by weight to exhibit the desired characteristics.

Attapulgite clay is sometimes used to make slurry when the groundwater is highly saline because attapulgite is less vulnerable to shrinkage and flocculation than bentonite under these conditions. However, attapulgites do not form stable filter cakes on the trench walls, and cannot typically be used in unstable granular soils (Millet, et al., 1992).

Cements

Cements are hydraulic binders, which form a progressively hardening paste that will set even under water. The main constituents of cement are anhydrous calcium silicates and aluminates. On hydration, cement converts to hydrated silicates and aluminates, and free lime. Crystallization of these hydrates is the origin of the set and hardening of cement.

Classification of cements according to the European Cement Association, and ASTM are provided in ICOLD (1985). Pure Portland cement is comprised of clinker (obtained by high temperature firing of a carefully batched mixture of clay and limestone) mixed with about 2 to 6 percent gypsum. Various other pozzolanic materials such as granulated slag and fly ash, and filler materials may be included.

Cements used in plastic conglomerates (self-hardening grout and plastic concretes) are batched at very low cement/water (C/W) ratios, on the order of 0.1 to 0.3. At these low C/W ratios, the strength characteristics of certain slag cements are much better than those of pure Portland cement.

Synthetic Bio-Polymers

Polymers used to make slurry muds are manufactured from organic compounds including bean curds, and guar-gum. These substances tend to degrade under long-term exposure to water, making them ideal for specialized applications. Day and Ryan (1992) for example report on the use of bio-polymer slurries for constructing granular drains. The primary application of polymers in situations where a watertight cutoff is desired is in situations where bentonite disposal is costly, or where the slurry is vulnerable to chemical attack. However, polymers do not form a stable filter cake, and are not recommended for trenches in unstable granular soils (Millet, et al., 1992).

10.3.2 Conventional Concrete

Composition

Highly impermeable concrete requires a large portion of cement to produce good workability and homogeneity. The intergranular voids must be completely filled with mortar, and all inert particles must be coated with a film of binding agent. To achieve these goals, the aggregate must be well graded, and have a maximum particle size of approximately 25 mm (1 in).

Special aspects of mix proportioning for concrete placed in slurry trenches are described in Xanthakos (1994). Fly ash or other pozzolanic materials are typically substituted for portions of the cement content to reduce cost.

Properties

For complete concrete placement through tremie pipes, the fresh material must satisfy the following conditions (Xanthakos, 1994):

- The mix must be flowable and have a plastic consistency. If the initial shear is to high, the flow is likely to be restrained, resulting in bentonite trapped in areas not reached by the mix.
- However, the mix must be cohesive enough to prevent segregation and bleeding. Concrete that bleeds or disintegrates under the pressure of its own weight can block the tremie pipe or accept bentonite.
- The mix should not set or stiffen too quickly but should remain workable until the pour is completed. The setting time must be extended to avoid adverse effects on concrete already delivered but not placed, or on sections placed but not completed because of delays.

10.3.3 Plastic Conglomerates

Plastic conglomerates include materials made from cement and clay, with or without aggregates, at very low cement/water ratios. This class of materials includes plastic concretes, soil-cement-bentonites, and cement-bentonites. As with conventional concretes, high impermeability, good workability, and homogeneneity are also important properties of the material.

Composition of Plastic Concretes

The materials most commonly used in plastic concrete include cement, bentonite, water, and aggregate. Fly ash and bottom ash constituents also may be utilized.

Bentonite is added via the fully hydrated slurry, usually of high viscosity (50 second Marsh reading). The bentonite serves to keep the cement grains and aggregates in suspension during placement, and to assure plasticity and impermeability. The percentage of bentonite typically varies from 2 to 12 percent by weight of water.
Cement/water ratios are typically very small, ranging from 0.1 to 0.3, depending on the type of cement.

Aggregates make up about 50 to 70 percent of the total volume of the mix. Aggregates should consist of clean, hard, strong, and durable particles, well graded and having a maximum particle size of about 25 mm (1 in).

Composition of Soil-Cement-Bentonites

The materials commonly used in soil-cement-bentonite include cement, bentonite, water, and soil aggregates. Fly ash and bottom ash constituents also may be utilized. This new type of backfill has been described by Dinneen, et. al.(1997). The material utilizes much of the native materials while still giving an impermeable, plastic product that is conducive to placement by many techniques and has some erosion resistant properties.

Cement usage is less that other plastic concrete mixes. Bentonite is added via a hydrated slurry at a rate of approximately 1 percent, by dry weight, of the soil aggregate. Cement is added at a rate typically between 4 and 10 percent by dry mass of soil aggregate. The soil aggregate is typically a well graded mix of gravel, sand, and soil fines. The introduction of soil fines in the aggregate gradation, typically in a percentage of 10 to 20 percent, is the principal difference between soil-cement-bentonite and plastic concrete. The soil gradation can be designed to meet general filter criteria with the surrounding foundation material to help limit erosion of the aggregate should a leak develop. While the inclusion of cement in the mix may inhibit the ability of the material to mobilize this filtering capability, some additional erosion protection is likely still realized.

Composition of Cement-Bentonites

Cement-bentonite is a mixture of water, cement, and bentonite to which is added set and hardening regulators. The average composition of these mixtures is as follows (per m^3) (ICOLD, 1985):

- 80 to 350 kg cement;
- cement/water ratio = 0.1 to 0.3, depending on type of cement; and
- 30 to 50 kg bentonite.

Mixtures thus proportioned will have densities ranging from 1.2 to 1.3 g/cm^3, and water contents ranging from 65 to 75 percent.

Properties

Strength and Deformability

The final strength and deformability of plastic conglomerates are influenced by the cement content, type of cement, and water loss by filtration. The need for high deformability requires a compressive strength as close as possible to the lateral constraint, but this strength must also be sufficient to resist soil stresses due to construction and use of the dam. Average strengths reported in ICOLD (1985) are 100 kPa (28 day) and 150 kPa (90 day) for cement-bentonites; and 1500 kPa for plastic concrete.

Xanthakos (1979) reports final set strength for cement-bentonite is usually in the range of 100 to 300 kPa, with modulus of elasticity ranging from 2000 to 5000 kPa, for mixes having 2 to 4 percent bentonite, 15 to 20 percent cement, and 5 to 10 percent aggregate.

Evans, et al, (1987) report shear strengths for cement-bentonite laboratory specimens ranging from 50 to 573 kPa for cement/water ratios of 0.06 to 0.4. Their test specimens contained fly ash in addition to cement. The specimens exhibited axial strains at failure ranging from 0.2 to 1.1 percent.

Evans, et al., (1987) report shear strengths for plastic concrete laboratory specimens ranging from 76 to 3427 kPa for cement/water ratios ranging from 0.12 to 0.73, on laboratory test specimens. Specimens exhibited axial strains ranging from 2 to 16.8 percent.

Dinneen, et al., (1997) report design strengths for soil-cement-bentonite to be approximately 700 kPa (28 day).

Millet et al, (1992) found that the cement/water ratio has a dramatic effect on deformability of the cement-bentonite backfill, with higher strengths associated with brittle behavior.

Permeability

Permeabilities of cement-bentonite backfills have been reported in the range of 10^{-5} to 10^{-7} cm/s, with typical values in the 10^{-6}cm/s range.

Permeabilities for plastic concrete range from 10^{-6} to 10^{-8} cm/s, with typical values in the 10^{-7} cm/s range. These hydraulic conductivities are approximately an order of magnitude lower than values reported for cement-bentonite, with decreasing hydraulic conductivities associated with increasing cement contents for both types of materials.

Permeabilities for soil-cement-bentonite range from 10^{-7} to 10^{-8} cm/s.

Durability

Durability of plastic conglomerates is considered with respect to strength and watertightness. In terms of strength, erodability tests provide basic guidelines for cement/water ratios as follows (ICOLD, 1985):

- for Portland cement, C/W = 0.2 to 0.25,
- for slag cements, C/W = 0.1 to 0.15.

Under normal groundwater conditions, permeability tends to decrease with time, probably due to clogging effects by soil fines. Under adverse groundwater conditions, including acidic or saline water, plastic conglomerate materials have been shown to resist permeability degradation.

10.3.4 Soil-Bentonite

Composition

Soil-bentonite is composed of soil aggregate produced from the excavation and bentonite slurry. It is usually advantageous to use slurry pumped from the trench rather than fresh slurry to prepare backfill. Bentonite content is typically a minimum of 1 percent of the total mix. Aggregates are usually well graded, and contain a significant percentage of fines, preferably plastic fines. Additional fines may be imported from offsite if needed to bring the fines content up to approximately 20 percent.

Properties

Permeability and strength of soil-bentonite backfill is dependent on proportioning of constituents, aggregate gradation, and slump of the backfill. The ideal backfill consistency for placement in the trench is a saturated paste having a low enough shear strength such that it flows easily, yet having sufficient stiffness to stand on a slope of about 10:1. This consistency corresponds to a slump cone value of about 2 to 6 inches (50 to 150 mm), and a water content of the mixture between about 25 and 35 percent (D'Appolonia, 1980).

Comparisons are often made between soil-bentonite and cement bentonite cutoffs, as these are common technologies used in the United States. The primary properties of concern are hydraulic conductivity, deformability, and strength. As a general rule, soil-bentonite walls will be more flexible, less permeable, and have lower strength than cement-bentonite walls. Thus, when large foundation deformations are anticipated beneath an embankment dam, cement-bentonite walls may be particularly vulnerable to cracking. However, soil-bentonite walls are more vulnerable than cement-bentonite walls to construction defects due to problems in backfill proportioning, mixing, and placement. Also, soil-bentonite walls may be more susceptible to hydraulic fracturing under high heads than cement-bentonite walls due to their lower strength. The use of soil-cement-bentonite backfill is an attempt to mitigate some of these concerns. The choice between soil-bentonite, cement-bentonite, and soil-cement-bentonite, or some other cutoff alternative should consider all design factors.

10.3.5 Materials for Other Types of Cutoff

In-Place Soil Mixing

The properties of the final wall produced by in-place mixing of soil and cement grout depends essentially on the physical properties of the soil. The basic property used for design and quality control is the compressive strength of the soil-cement mix. For sands and clays, the 28-day compressive strength has been found to range between about 500 and 2700 Pa and is almost twice the 7-day strength. Within the working stress range, the material behavior is linearly elastic. For design purposes, the shear strength can be assumed as 1/3 the unconfined compressive strength (Xanthakos, 1994).

Taki and Yang (1989) reported hydraulic conductivities for soil-cement walls on the order of 10^{-6} to 10^{-8} cm/s.

Injected Grout Screens

Grouting materials used in injection systems fall into three distinct categories: suspensions, solutions, and emulsions. Suspensions include bentonite slurry, cement-bentonite, or cement-filler formulations. Chemical solutions include precipitate grouts such as sodium silicate gels and patented polymers. Asphalt emulsions designated as ASPS mixes are also used. ASPA mixes and chemical grouts are considerably more expensive than cement/bentonite mixes, but are also less viscous on injection, more impermeable after set, and highly resistant to acids, salts, and other chemicals. On some projects, it is feasible to conduct grout injections in two stages, using a low-cost cement/bentonite grout to initially fill large voids, followed by chemical grouting to penetrate smaller void spaces.

Cement-bentonite mixes having 5 to 7 percent bentonite and a cement/bentonite ratio of about 2:1 are generally optimum in terms of low permeability, flexibility, and erosion resistance. Lower cement/bentonite ratios are used when impermeability is the primary requirement. Permeabilities in the range of 10^{-6} to 10^{-8} cm/s can be achieved, with the lower values associated with formation of an adequate filter cake (Leonards, et al., 1985).

The penetrability of the grout depends on the pressure gradient, soil permeability, grout viscosity and shear strength, and grout particle size. The tendency of the grout to extrude under pressure head is resisted by the shear strength of the mix. A shear strength of about 0.07 to 0.14 kg/cm^2 (1 to 2 lb/in^2) can resist hydraulic gradients of 100 in soil having an average grain size of 25 mm (1 in) (Xanthakos, 1979).

Interlocking Geomembranes

Synthetic geomembrane materials comprise numerous polymer and rubber-based formulations including, polyvinyl chloride (PVC), low-density polyethylene (LDPE), high-density polyethylene (HDPE), chlorinated polyethylene (CPE), chlorosulfonated polyethlene (Hypalon), and butyl rubber. These materials are flexible, impermeable, and highly resistant to chemical attack. Engineering characteristics and properties of these materials are described in detail by Koerner (1990).

10.4 QUALITY ASSURANCE AND QUALITY CONTROL

Controlling the quality of the cutoff wall is accomplished in two major areas - wall construction techniques and backfill material manufacturing.

First, the construction techniques used for excavation, cleaning of the wall, and placement of backfill are all critical in obtaining a quality product. Since walls for dams are usually not able to be directly inspected once installed, quality assurance relies most heavily on monitoring of the construction on a real time basis. Quality of construction is the primary contributor to a quality wall.

Some post construction testing of the wall by core drilling can be performed to inspect the quality of the backfill material; inspect the joints between panels; or to investigate for suspected pockets of deleterious material. Obtaining quality cores can be difficult to obtain in the softer backfill materials. Inspection of cores can often lead to ambiguous conclusions.

In situ permeability testing can be performed in holes drilled in the constructed wall. The tests are usually difficult to perform especially if packers are to be set in the softer backfill materials. Results are difficult to interpret and there is concern that damage to the wall may be caused by the water pressures used in the tests or by the process of inflating the packers.

In all cases of post-construction testing, the amount of the wall tested is only a small fraction of the total.

Monitoring the quality of construction can be a time consuming, exacting process depending on the type of construction used. Continuously excavated walls are much easier to monitor than panelized walls. Construction items monitored can include initial wall alignment; foundation materials being excavated; equipment alignment during and excavation; completed excavation alignment and dimensions; cleaning techniques; slurry properties during excavation and prior to backfill; and backfill placement techniques. All of these items require nearly constant surveillance during construction.

Some of the key material properties for controlling the quality of the slurry and backfill material have been discussed above. All of these can be tested by standard testing procedures. Again, placement techniques are critical in guarding against such things as segregation of the material matrix during placement and inclusions of slurry pockets during backfilling.

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CHAPTER 11 — CONSTRUCTION ISSUES

11.1 INTRODUCTION

Construction of an embankment dam is the execution of the site specific design for that dam. Considerable additional information is obtained about the foundation and fill materials from required excavations for the embankment and appurtenant structures. Operations in the borrow areas and processing of materials provide further data about the characteristics of fill materials. Data from the quality control program for fill construction provides information on insitu fill conditions. Designers must reassess the design concepts and assumed conditions in the light of the actual conditions observed at the construction site. Reanalysis of stability conditions and redesign of various aspects of the foundation treatment, embankment zoning or construction procedures may be required. The design of an embankment dam continues through the construction and a close coordination between design and construction engineers is essential. Arthur Casagrande is attributed with saying that the design of an embankment dam is not complete until the reservoir has been full for five years.

11.2 CONSTRUCTION ISSUES

11.2.1 Quality Construction

A well-designed embankment dam makes the best use of local materials to fit site conditions. Quality construction is necessary to transform the design concepts into a successful project. ICOLD Bulletin 85 addresses the responsibilities of the various participants in the construction to assure success. The following is an excerpt from the Bulletin:

> *"Successful construction projects have several common characteristics which key their success. It is particularly important to note that these projects generally have very good construction specifications/drawings and contract documents, which are well engineered, clearly and completely presented, and based upon solid design data and a comprehensive and thorough site investigation program. All the requirements are presented in the specifications; the requirements are specific, reasonable, and attainable; and these requirements are supported by a fair payment schedule which allows the contractor to be paid appropriately for the work performed. The participants are knowledgeable enough to expect the possibility of unknown conditions. The contract documents provide a fair and equitable means to deal with the unknowns and changes, problems are resolved promptly, and the project is well administered and funded to cover these unknowns.*
>
> *"In support of the contract and specifications documents is a strong working relationship between the participants. This relationship between the parties permits clear and open communications, the development of trust, and maintenance of confidence. Each project member is supported by experienced*

and knowledgeable personnel with the authority and commitment to deal with each other in good faith. Quality construction is fundamentally a team effort."

11.2.2 Attention to Details

The details which assure successful implementation of the design at the site deal with:

- conditioning of the material in the borrow
- material placement and compaction procedures on the fill
- cleanup, shaping, and treatment of the foundation embankment interface
- selection of special material for placement at the interface
- initial placement and compaction of fill onto the foundation
- placement and compaction of fill against structures
- actions to be taken if less favorable or even unfavorable conditions are encountered
- limits on material variability
- protection of the constructed portions from adverse weather
- selection of construction equipment
- impact of construction schedule constraints, etc.

All of the above are detail items to plan and arrange for in advance. Preparation of designs and instructions for details such as these is most important to ensure long-term reliability of the completed dam. Time must be spent in identification of the details and in describing by words and sketches the procedure to implement them. These details are transferred to the construction by way of the drawings and specifications. The design intent, construction cautions, and options should also be conveyed to the construction staff. This can be done with a separate document or written into the specifications, and involving the design staff at key phases of the construction.

11.3 CONTROL OF WATER

Inadequate control of ground water seepage and surface drainage can cause major problems for compacting fill against the foundation. Erosion of foundation slopes by surface drainage may result in deeply rutted surfaces and eroded material being deposited below the slope and on the foundation. Both of these conditions will prevent proper placement of embankment materials. The ruts and deposits must be removed prior to embankment placement. Interception of runoff with ditches and dikes, and drain ditches to lead water away from the construction will mitigate adverse erosion.

It is impossible to compact impervious fill where groundwater seepage exits through rock fractures or where water ponds on the foundation. Flow must be controlled at the area of fill placement and at the source, if the source is higher than fill placement. Pipes imbedded in the source and standpipes sealed against the foundation are measures commonly employed. Sumps, ditches, and dewatering systems may be used to control water flow from interfering with embankment placement. Ditches should never extend beyond the center of the core in either the upstream or downstream direction. All pipes, ditches and sumps must be completely sealed by a grouting system when no longer needed. The water level in soil foundations and in the embankment must be maintained at least three meters below the level of embankment construction. Further discussion and details are available in USBR (1991).

11.4 BORROW AREAS

Planning of borrow area operations to provide well blended material at the moisture content for proper compaction enhances construction efficiency and promotes good quality compacted embankment. Conditioning of fill material is generally done more efficiently and effectively in the borrow area. Conditioning refers to adjustment of the moisture percentage and uniformity, and removal of oversize rock. To do the conditioning on the fill will normally require more equipment to operate in the often already confined space, increase the time for the fill placement-compaction process, and increase the potential for material nonhomogeneity. When constructing in a dry climate, a good rule to follow for addition of water to a layer on the fill is to only allow the addition of water to compensate for moisture evaporated during the construction of the layer. In wet climates, borrow area surface drainage and excavation on a vertical face may be effective to prevent material from getting excessive moisture. Blending of several soil layers in a borrow pit may be accomplished by excavating a vertical face with a power shovel, front-end loader, vertical cutting self loader, or a wheel excavator or with scrapers loading on an incline. Loading scrapers uphill on an incline is more effective for mixing than loading downhill. There are other schemes for blending materials in special cases, USBR 1991.

11.5 FILTER/DRAIN ZONE PARTICLE SEGREGATION

Concern with segregation is primarily with the downstream filter zone material and that larger particles are being deposited along the impervious core-filter interface. This creates voids into which core fines can migrate. Prevention of segregation can be by design of the gradation and by control of the construction methods. For example, at the processing plant, stack the material using a ladder to preclude the larger particles from rolling to the bottom of the slope; load the material to preclude getting mostly segregated particles; and discharge the material onto the fill in smaller piles for spreading, or belly dump trailers placing in windrows to minimize dozer spreading distances.

Also of concern are cross-overs, where the hauling equipment has disturbed the impervious and filter zones to reach the upstream and downstream shell zones. These disturbed areas must be excavated, removed, cleaned and replaced to ensure that no damaged fill remains and the impervious and filter zones conform to specifications.

11.6 COMPACTION UNIFORMITY

Uniform compaction of the impervious fill maximizes strength and impermeability, and minimizes the potential for differential settlement and formation of high permeability paths through the fill or along the foundation fill interface. Uniformity of compaction is easily achieved when well maintained, specified equipment are used in an orderly work sequence. For example, an agricultural disk or harrow is used to break up equipment tire-compacted material; a motor patrol keeps the fill level; hauling equipment only runs on the previously placed lift not yet ready for new lift placement; dozers spread the material uniformly to the specified loose lift thickness; a water wagon sprays a uniform water spray over the lift if the material has dried; a harrow or disk passes through the loose lift to break up clods and provide mixing; and the compactor makes the required number of full coverage passes. An organized fill placement equipment spread reduces costs and maximizes the development of desirable engineering properties of the fill. When areas of the fill are not fully compacted or the material is too dry to allow full compaction, that area will settle upon wetting by the seepage front passing through the embankment as the reservoir fills. Strong layers above may bridge the area and a high permeability channel is formed with the potential for piping.

The full compactive effort of a compactor will only be realized if the cleaner bars are maintained to fully clean material from between the tamping feet, Figure 11.1. Compactors operated at too high speeds will not effectively compact the fill or shears may develop. Tamping feet worn short reduce the compactive effort and tamping feet worn into points push the material to the sides, also reducing the compactive effort. Worn harrow or disk blades don't mix or scarify deeply enough. Fill surfaces kept level enhance equipment efficiency.

11.7 COMPACTION AT STRUCTURES

Of great importance are construction issues regarding structures passing through an embankment dam or against which the embankment is abutted such as outlet works, conduits, and spillway structures. Construction methods must preclude internal erosion or piping along the structural surfaces against which and around which embankment must be compacted. Earthfill material must be specially selected to have a higher plasticity, proper moisture for special compaction by hand operated, wheel rolled or other small equipment, and maximum particle size of about 1 inch (25 mm). Excavations should be adequately sized to accommodate specified compaction equipment. Material must be placed in thinner layers than the normal embankment and with a surface sloped to the structure. All placement and compaction should be done in daylight to allow maximum visual inspection. A higher frequency of testing (number of tests/volume placed) than for the normal embankment must be used to control inplace unit weight and moisture content.

11.8 WEATHER PROTECTION

Maintaining the fill level or with a slight slope up at the abutments precludes water from ponding and softening the fill. Sealing the fill surface before it rains is also necessary. Maintaining the filters higher than the core may preclude contamination from muddy water flowing over filters. Winter frost protection may be a sacrificial fill cover which is removed in the spring. The installation of frost tubes will indicate the depth of frost penetration and the depth to which fill testing needs to be made to decide how much material should be removed for spring start-up.

11.9 QUALITY CONTROL

The quality control program to assure attainment of the quality standard required for the construction of fill dams consists of two separate and specific aspects; quality assurance and quality control, ICOLD Bulletin 56. Quality assurance provides validation of the standards and procedures to be used, and oversees the implementation of the quality control plan. Quality control defines the standards and procedures for measurement, execution of the procedures, and the determination and enforcement of the quality standard. It consists of inspection (visual examination, measurement and testing) and full documentation of all the methods and equipment used and test results obtained.

In addition to the daily visual inspections by the inspector on the fill, it is prudent to excavate test trenches in the fill to examine the embankment and to test the fill at a greater depth than for the usual quality control tests. When the fill is about a meter above the foundation, a test trench is excavated to check the bond with the foundation and examine the uniformity of material layers. Layers should not be identifiable or only with great difficulty. Similar trenches against the abutments are also appropriate. Test trenches are repeated at elevation intervals as the embankment rises. Trenches excavated in a T- or L- shape improve inspection access to look for unbonded layering and laminations at the re-entrant trench corners.

11.10 CONCLUSION

It is essential that we make the most thorough investigations that are practicable, prepare our designs and construction for the most probable set of conditions indicated and plan actions to take if conditions are less favorable. In the construction of an embankment we are afforded one excellent opportunity to improve or correct conditions or procedures and that is when they are noticed. To not correct a condition even though it appears minor, is a major error. It is generally not the single flaw not removed which creates the major problems, but the unfortunate combination of flaws not discovered and those not corrected which develop into the major problem.

> *"No mistake in designing and building a dam is permissible. Design and construction go hand in hand . . . Every dam should be the best Rolls-Royce and not a fliver . . . (the engineer's) duty does not lie only in saving a maximum of his client's money. It demands absolutely that the public be afforded a maximum of safety. If the client is unwilling or unable to pay for that maximum then he should not have that dam."* (Thaddeus Merriman, 1939)

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