EARTH DAMS: GEOTECHNICAL CONSIDERATIONS IN DESIGN AND CONSTRUCTION

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Submitted to the Faculty of the Graduate College
of Oklahoma State University
in partial fufillment of the requirements
for the Degree of
DOCTOR OF PHILOSOPHY
December, 1984

EARTH DAMS: GEOTECHNICAL CONSIDERATIONS IN DESIGN AND CONSTRUCTION

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PREFACE

This study is concerned with the analysis of the geotechnical considerations for the design and construction of major earth dams. The objective was to develop and present empirical design procedures from a review of existing earth dams. The majority of the dams studied were designed by, and constructed under the direction of, the Corps of Engineers. Also, to provide diversity, dams of the Bureau of Reclamation and of private interests were included.

The author wishes to express his appreciation to his major adviser, Dr. Donald R. Snethen, for his guidance and assistance throughout this study. Appreciation is also expressed to the other committee members, Dr. James V. Parcher, Dr. Richard N. DeVries, and Dr. Douglas C. Kent, for their invaluable assistance in the preparation of the final manuscript.

A special note of appreciation is extended to Mr. Bill Erdner of the Corps of Engineers for his encouragement and assistance throughout this endeavor. This appreciation is also extended to the many people throughout the Corps of Engineers who provided engineering data for this study.

Finally, special gratitude is expressed to my wife, Lois, for her patience and understanding.

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CHAPTER I

INTRODUCTION

The design and construction of Fort Peck Dam in Montana, Figure 1, which was completed in 1950, initiated the final phase of modern earth dam engineering in the United States. Fort Peck Dam was one of the first major earth dams designed by, and constructed under the direction of, the U.S. Army Corps of Engineers. The Corps of Engineers had been charged with the responsibility of river improvement and flood control along the Mississippi River, and in 1936 their responsibility was enlarged to include flood control activities nationwide, including construction of major dams (134). Fort Peck Dam was one of the first major dams to be constructed for the purpose of flood control; more appropriately, Fort Peck Dam was one of the first major dams to be constructed as a multiple-purpose project incorporating other reservoir purposes along with flood control.

In itself, the engineering experience attained at Fort Peck continues to be incorporated in new dam designs. Relief wells to control foundation seepage and reduce uplift pressures were first developed and installed at Fort Peck. Liquefaction, as a phenomenon, became understood through stabilization of slope slides along the embankment.

Prior to the entry of the Corps of Engineers into dam engineering, the Bureau of Reclamation of the Department of the Interior had been

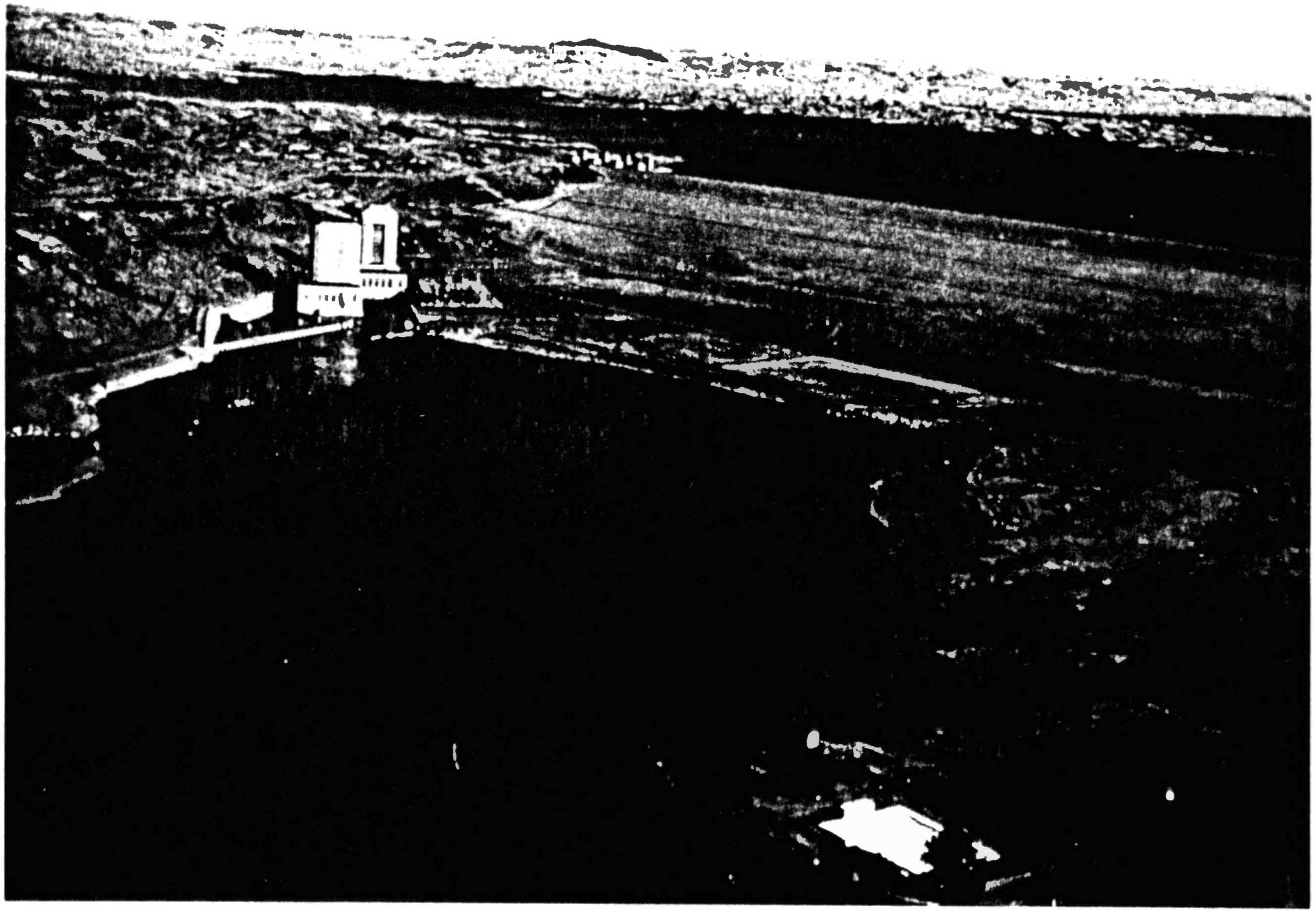

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Figure 1. Fort Peck Dam, Missouri River, Montana.

developing irrigation projects in the western states. The first earth dam designed by, and constructed under direction of, the Bureau of Reclamation was Belle Fourche Dam in South Dakota and was completed in 1908 (211). Because early Bureau dams were single-purpose projects for reservoir storage, the design and construction of the dams provided for near-elimination of foundation and embankment seepage and did not incorporate seepage control features of flood control dams. Newer Bureau dams are multiple purpose projects and provide features similar to Corps dams.

As the experiences gained from the design and construction of Fort Peck Dam have been incorporated into the design and construction of new earth dams, a vast quantity of experience is available from the design and construction of other earth dams. However, because of the quantity of information and, unfortunately, the reluctance of many dam engineers to publish and distribute the information, much of the knowledge gained through experience is lost.

The practices of earth dam engineering of the Corps of Engineers and the Bureau of Reclamation are foremost within the United States. The study of these practices supplemented by engineering data from the Tennessee Valley Authority and the Soil Conservation Service, as well as from private engineering companies such as Harza, Stone and Webster, Shannon and Wilson, and Tippetts-Abbett-McCarthy-Stratton, provides the basis of the empirical design of earth dams. And, in spite of advances in soil mechanics theory and digital computers, the engineering of earth dams will remain predominantly empirical.

CHAPTER II

INITIAL CONSIDERATIONS

Conditions Influencing Design

Site conditions have a greater influence on the design of an earth dam than corresponding site conditions have on other engineering structures. In addition to site-specific foundation and environmental conditions, the earth embankment is constructed almost exclusively from onsite materials. In addition to providing bearing capacity, foundations must also provide, or be treated to provide, erosion and seepage resistance. Environmental conditions, particularly climatic conditions, influence the design through past activity by weathering of foundation and borrow sites; present activity by modification of compaction water content; and future activity by erosion and wave action on embankment slopes. Climatic conditions have influenced design policy concerning compaction of embankment fill in that the Bureau of Reclamation, operating within the arid western portion of the United States, is adamant about compacting dry of the optimum water content of the embankment material.

Administrative policies of design agencies have a significant influence on the design of earthfill embankments. Originally, the Corps of Engineers established individual districts for the design and construction of major earth dams. Two good examples are Fort Peck Dam in Montana and Denison Dam in Texas, designed and constructed from about 1935 to 1950. This policy changed in the late 1950's to provide regional design offices, as districts became more regional during the period of major dam construction which lasted until about 1970. Unlike earlier earth dams, which were very site specific in design, these later earth dams tended to be somewhat less specifically suited to a unique site and were to some extent packaged designs fitted to the sites. At the present time technical expertise within the Corps of Engineers has diminished to the point that only a very few district offices have sufficient technical expertise to design a major earth dam. Furthermore, efforts have been initiated to provide central design offices for areas which incorporate many engineering districts. Ironically this centralization of design capability within the Corps of Engineers is concurrent with the decentralization of the design capability of the Bureau of Reclamation. All major design functions were undertaken by the Bureau at their main office complex in Denver, Colorado, and this practice was critized as having contributed somewhat to the failure of Teton Dam in Idaho (29). Consequently, the Bureau now establishes a resident engineer office at each damsite to provide all design and construction inspection for that dam.

Within the United States, the construction procedures related to rolled-earthfill dams and rolled-rockfill dams have become so cost effective through standardization as to be prevalent. The cost effectiveness of compacted earthfill using modern earthmoving equipment eliminated hydraulic-fill embankments, and is conincidental with the problems of low-density material inherent in hydraulic fills (57). In some countries hand labor may be more cost effective than equipment, or more politically expedient: in China, for example, cutoff trenches are occasionally excavated by hand (15).

Conditions prevailing at damsites provide the major influence in the design of earth dams. Consequently, thorough explorations are required to determine the characteristics of the foundation and the characteristics and extent of borrow material for embankment construction.

Foundation Characteristics

An earth dam can be constructed on any foundation as evidenced by Willard Dam, "the dam with only one abutment and no bottom," which was constructed by the Bureau of Reclamation on lakebed sediments of Great Salt Lake in Utah (214). Foundations of low shear strength soils, of which the foundation for Willard Dam was an extreme case, require stage construction (used at Willard Dam), flatter embankment slopes, or other means to maintain stability of the structure. The settlement of compressible foundation soils may require camber to compensate for future loss of freeboard, or may require special design features to eliminate cracking of the embankment or reduce the adverse effects of cracking.

The type of positive cutoff through a pervious foundation, of either soil or rock, may influence or dictate the location of the impervious core or impervious membrane of the embankment. This is particularly true of slurry trench cutoffs and concrete cutoff walls through foundation soils which are best located at or beyond the upstream embankment because of the dissimilarity of the cutoff material from the foundation. For earthfill cutoff trenches, the location of the embankment core influences the location of this foundation cutoff.

The absence of a positive cutoff through a pervious foundation may require thicker horizontal filters within the embankment, and the gradation of horizontal filters is partially dependent on the gradation of foundation soils.

Special construction procedures are required to properly bond the embankment core to foundation rock to prevent localized leakage at the contact. Excavations which expose shale within the foundation require special construction sequencing in order to minimize the duration of exposure and thereby minimize slaking of the shale. Special attention should be directed to stratified foundation rocks, such as shales, schists, siltstones, and claystones (37). Flattened embankment slopes and wide loading berms may be required to maintain embankment stability against sliding within foundation rock.

Foundation treatment is part of the overall design of the project and can be influenced by conditions which influence embankment design. A positive cutoff may not be necessary, for example, if authorized reservoir purposes do not consider underseepage as an economic loss.

Characteristics and Availability of Borrow

Construction materials for an embankment dam are readily available at the site, or within reasonable haul distances. This availability of embankment materials, along with constructability on any foundation, caused the decline of construction of concrete dams within the United States.

The characteristics of the available construction materials have perhaps the greatest influence in the design of the dam embankment. More precisely, the quantity of impervious material, either as a proportion of the overall materials quantity or as a percent content of the materials themselves, contributes the greatest influence. The volume of the impervious core relative to the total volume of the embankment is very nearly equal to the quantity of impervious material relative to the total quantity of available borrow. This is a general statement and may be violated by a few specific examples of disagreement. The volume of impervious core relative to the total embankment volume ranges between 0 percent for embankments with special upstream membranes and constructed from borrow areas devoid of impervious material, to 100 percent for homogeneous embankments constructed from borrow areas containing only impervious material.

The economic advantage of an earth dam is related to the availability of suitable construction materials. Processing of large quantities of borrow materials to improve their suitability for use as embankment fills serves to eliminate this economic advantage. Bentonite has been mixed into pervious soils to provide impervious membranes for sewage lagoons and landfill pits. Because of the enormous quantity of impervious material required in an earth dam, the expense of the bentonite, coupled with the processing expense, would be prohibitive for mixing with pervious soils to provide core material. Processing of small quantities of available impervious soils into pervious soils has been accomplished on a few earth dams. At Howard A. Hanson Dam, constructed by the Corps of Engineers in Washington in 1962, silty sand topsoil was blended into a clean, well-graded sand and gravel mixture to reduce the permeability of the coarse material in order to use it as impervious material (83).

The diameter of the maximum particle size is generally limited to the lift thickness in order to facilitate spreading and accommodate compaction. Screening may be required, or offered as a construction alternative, for materials containing an appreciable amount of oversized cobbles. For materials containing only occasional cobbles, removal may best be accomplished by hand labor or mechanical rakes. At San Antonio Dam, constructed in California by the Corps of Engineers in 1956, nesting of cobbles were observed where lifts were joined from opposite directions; continuous spreading in one direction eliminated this problem (68).

Considerable quantities of material may be obtained from required excavations. The quality of the required excavation may allow placement within the impervious core or within outer shells of the embankment. Materials unsuited to critical embankment zones, due to lower shear strengths or weathering potential, may be incorporated into the embankment as random zones. For central core embankments on deep soil foundations, the best location of these random zones for less ideal materials is within the downstream shell (125). The slip surface of the most critical stability arc does not pass through the outer shells; and the downstream shell, behind the vertical filter and above the horizontal filter, should never become saturated. Waste material may be placed along the upstream slope below riprap slope protection to protect the embankment during initial filling.

Climate

The Bureau of Reclamation specifies compaction dry of the optimum water content in order to eliminate stability problems associated with high pore pressures (27). The Corps of Engineers advocates compaction at water contents which are practicably obtained (128). These distinctly different policies regarding compaction water contents are the result of the climate in which each agency performs its task. The Bureau has designed and constructed earth dams predominately within the arid West; while the Corps has designed and constructed dams in the more rainy Midwest, the West Coast, and the East.

Homogeneous clay embankments constructed during rainy seasons may exhibit considerable lateral movement and bulging of the embankment (219). At Howard A. Hanson Dam in Washington the blended core material of silty sand and well-graded sand and gravel enabled satisfactory construction of an earth dam in an area of high precipitation (greater than 80 inches per year) (83). Only a very a few days of construction time were lost due to weather or waiting for material to dry, and these delays were caused by short periods of high precipitation.

After completion, earth dams are not readily damaged by weather conditions, and it is not necessary to provide special design details to protect them (57). A significant exception is the construction of earth dams in areas of permafrost. Substantial measures along the upstream slope and upstream portion of the foundation must be employed to prevent seepage infiltration into the earthfill. The earthfill should be dry to prevent heaving of the fill as the level of the permafrost is raised because of the insulating effect of the embankment over the existing permafrost. Coarse gravel is used to envelope the earthfill and provide drainage and aeration. Aeration allows the dam to be "overcooled" in winters to prevent thawing during short artic summers (39). Figure 2 shows a typical embankment section for construction of an earth dam in areas of permafrost.

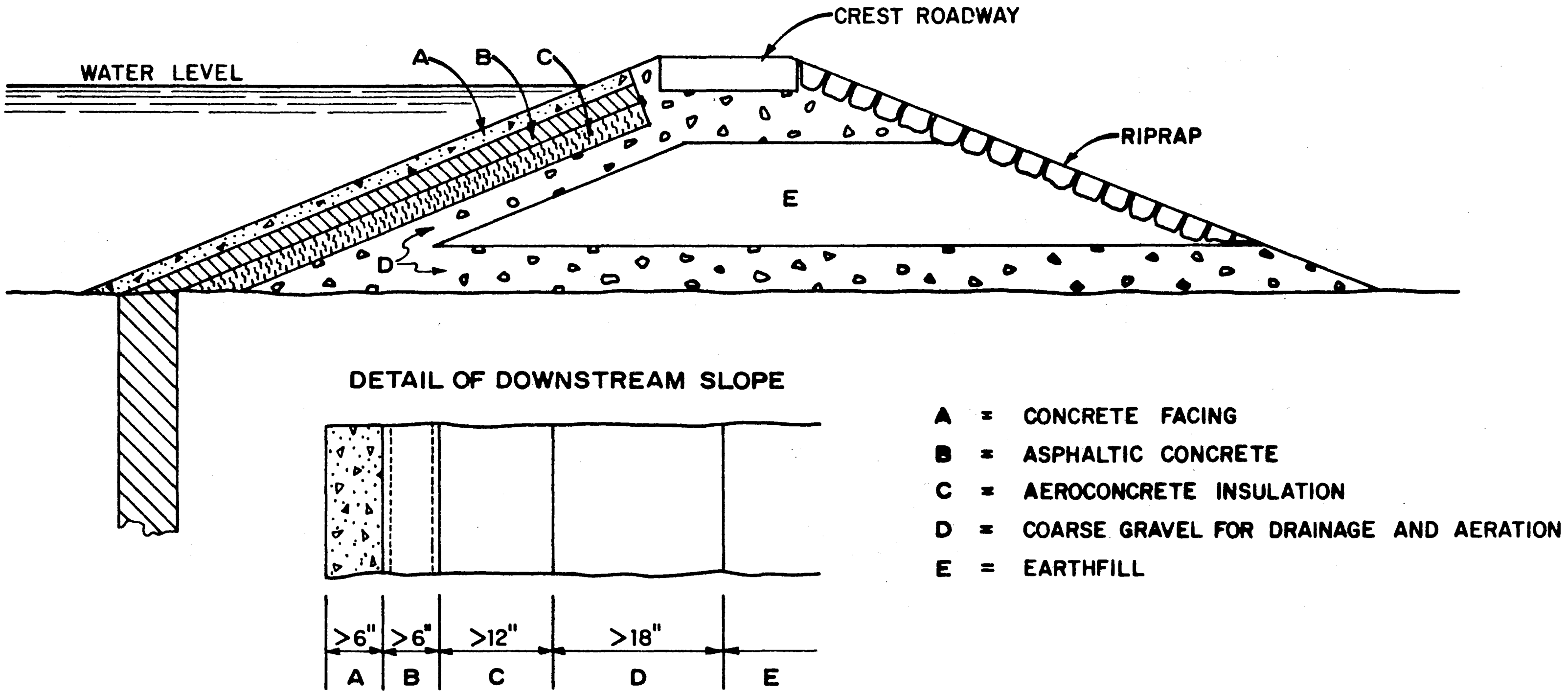

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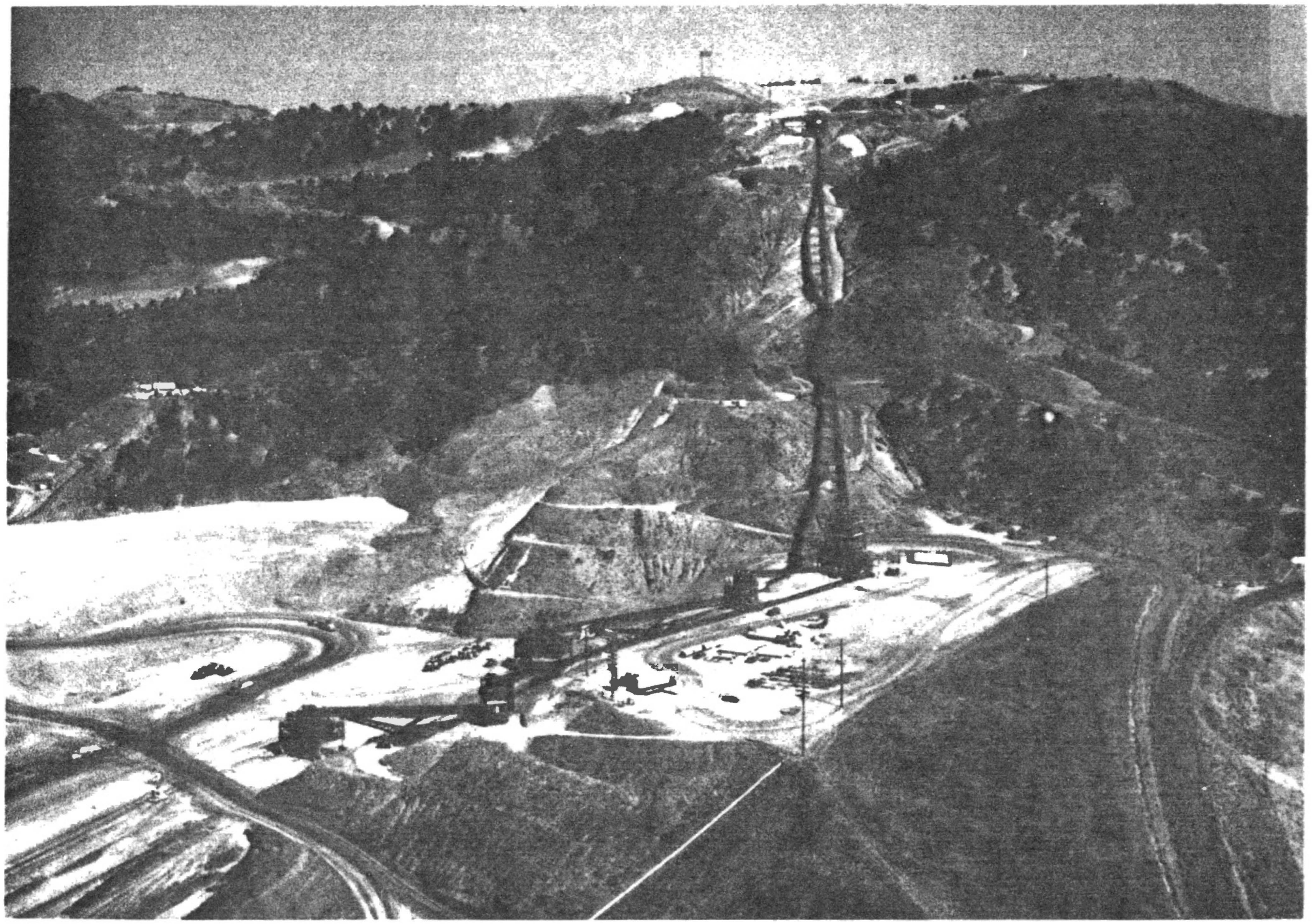
Figure 2. Earth Dam in Areas of Permafrost.

Valley Topography

The site requirements pertaining to the topographic character of a valley are such that the valley should provide a water tight basin of adequate size, a narrow outlet to permit economical construction of a dam, and a suitable location for an adequate spillway for surplus waters (11).

A constriction in a valley is desirable in order to limit the length of the embankment but it may not provide the best damsite if the foundation is problematic. Any potential damsite must be thoroughly investigated through detailed geologic explorations and interpretations. In narrow valleys with steep rock walls, special abutment treatment is required to reduce leakage at earthfill-abutment contacts and to reduce adverse effects of differential settlement. Wide valleys with gently sloping abutments present very few design and construction problems.

Haul roads into deep, narrow valleys from terrace and ridge borrow sources may be prohibited for safety reasons and environmental impact concerns. At Abiquiu Dam, constructed by the Corps of Engineers in 1963, borrow material was transported to the embankment by a belt conveyor system. The borrow area was upstream of the damsite at a widening of the valley. The conveyor system provided moisture control of the fill and scalping of oversize materials (82). At Warm Springs Dam, constructed by the Corps of Engineers in 1983, borrow material was transported by a belt conveyor system from beyond the left abutment, into the valley, and onto the embankment fill, as shown in Figure 3. This conveyor system also provided moisture control and scalping of



Source: (123)

Figure 3. Conveyor System for Borrow, Warm Springs Dam, California.

oversized materials. The conveyor system at Warm Springs transported the material down the valleyside, a difference in elevation of about 300 feet, and the polarity of the electric motors which supplied initial power for the conveyor could be reversed changing the motors to electric generators. Electrical power for the project was generated during construction by the borrow material moving downgrade (123).

Steep valley walls that tower over the reservoir should be fully investigated to determine any potential of sliding. Massive reservoir slides displaced water over Vaiont Dam in Italy in 1963 and claimed 3000 casualties. The slides were precipitated by excessively heavy rains in the reservoir area (47). At New Melones Dam in California, constructed in 1979 by the Corps of Engineers, the reservoir rim was investigated for potential slide areas during design, and potentially unstable areas were treated during construction by flattening the slopes or removal of unstable material (122).

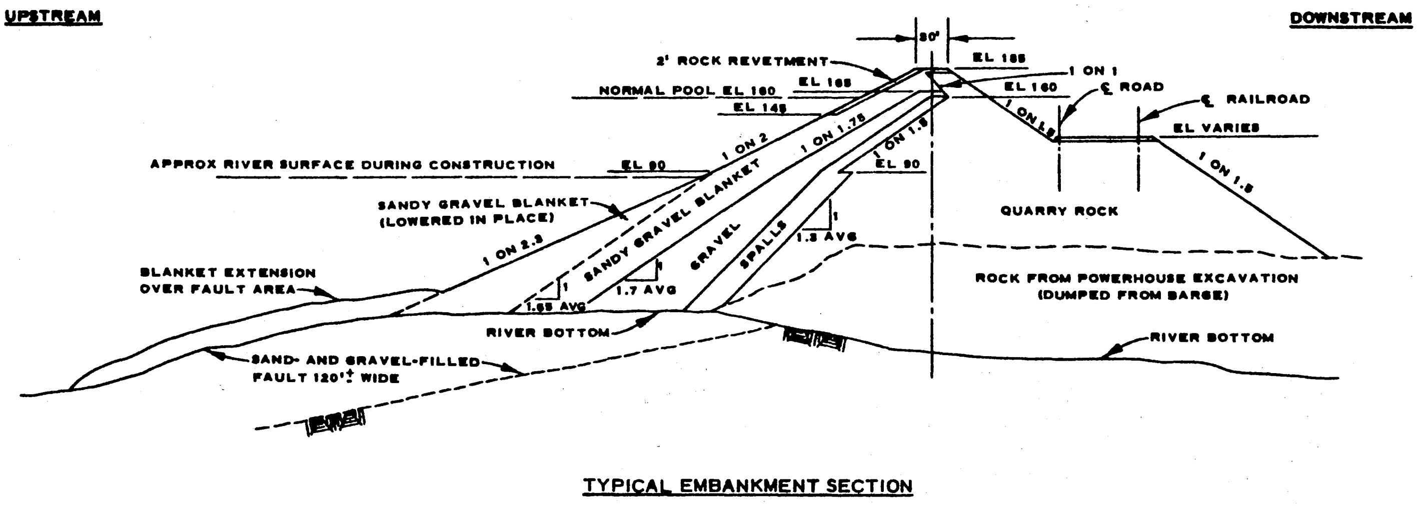
Stream Diversion

Potential problems involved with stream diversion during construction of an earth dam, and their influence on design, are largely dependent upon the width of the valley at the damsite and the volume and seasonal variation of streamflow. Stream diversion into the outlet works conduit at Optima Dam in Oklahoma was ceremonious only owing to an absence of water in the Canadian River. At the other extreme, the Dalles Closure Dam, which is a 2,000-foot long dam across the Columbia River in Oregon and completed in 1957 by the Corps of Engineers, was constructed without dewatering the foundation. The lower portion of the embankment was constructed by dumping a massive downstream rock section into 80 feet of water flowing at 200,000 cfs (147). The unique typical embankment section of the Dalles Closure Dam is shown in Figure 4. As a result of construction of the rock section, the Columbia River was diverted through the control structure.

Within deep, narrow valleys, stream diversion into outlet tunnels or into specially contructed diversion tunnels is necessary prior to foundation treatment in the valley floor. At Warm Springs Dam, the stream was diverted through the outlet works tunnel in the left abutment prior to construction of an upstream cofferdam across the valley (123).

Within wide valleys, a substantial portion of the embankment can be constructed prior to stream diversion through the outlet works conduit or tunnel. The final portion of the embankment, the closure section, can then be constructed behind an upstream cofferdam which provides a barrier in the event of rising waters within the reservoir area. All foundation treatment can be completed prior to final diversion by temporarily diverting the stream over a completed portion of the foundation in order to provide treatment for the final foundation segment. At Stockton Dam in Missouri, constructed in 1970 by the Corps of Engineers, temporary stream diversion was made through an excavated channel over prepared foundation so that rock treatment and grouting could be completed before final diversion (111).

The closure section is constructed quickly because of the short length of the section. If the closure section is constructed on foundation soil, sufficient settlement may occur to cause cracking of the embankment. All significant settlement of the previously constructed portion of the embankment could be expected to occur prior



Source: (147)

Figure 4. The Dalles Closure Dam, Columbia River, Oregon.

to construction of the closure section. At Kaw Dam in Oklahoma seepage was observed along the downstream slope after reservoir impoundment, however, piezometer data indicated that the impervious core was intact. The seepage was finally concluded to be surface infiltration percolating along the contact slope between the embankment proper and the closure section. This was confirmed through correlation with rainfall data (170).

The potential of cracking within the closure section which results from segmented construction causes a significant influence on the design of the embankment and on construction sequencing of the contract. Specifying construction of the main portion of the embankment to an intermediate height prior to closure has been successful at a number of dams. However, these dams were usually constructed using stage construction techniques in order to increase foundation shear strengths which made the intermediate height seem more plausible and thereby making the contract easier to control. Special downstream filters or increased thickness of downstream filters can provide defensive design measures against potential cracking, as was done at Garrison Dam in South Dakota.

Measures may be implemented which better enable the embankment to resist damaging effects as a result of overtopping during construction. At Corin Dam in Australia, mesh reinforcement was used to prevent unraveling of rockfill in the event of overtopping. Allowing a greater frequency of overtopping permitted a smaller diversion channel to be used. The mesh was removed after completion of the embankment (44).

Seismic Activity

The possible modes of failure of earth dams during earthquakes include loss of freeboard, piping through cracks induced by ground motion, and overtopping of the dam. Loss of freeboard may result from differential tectonic movements, slope failures due to ground motion or sliding within weak foundation material, and consolidation of embankment and foundation materials (53). Piping may occur as a result of transverse cracking of the embankment due to ground motion, differential tectonic movements, or major disruption of the dam through fault movement in the foundation (52). Overtopping of the dam, with no appreciable settlement of the crest, may result due to seiches (earthquake-induced water waves), reservoir slides, or failure of the outlet works and spillway (51).

Design considerations due to potential seismic activity are not, for the most part, analyzable problems and are handled through defensive design measures.

Project Purpose

The major influence on design of the dam embankment as a result of the project purpose, or purposes, is the economic value of potential seepage through the embankment and of potential underseepage through the foundation. If authorized project purposes include water supply, irrigation, or hydropower, the economic value of seepage and underseepage may justify extensive measures to significantly reduce water loss from the reservoir. If, however, project purposes are exclusively flood control, sediment storage, or aquifer recharge, only control of seepage and underseepage is important and reduction of seepage quantities is an insignificant consideration.

With very limited exception, dams of the Bureau of Reclamation have been provided with impervious cores or membranes for seepage reduction through the embankments and with positive underseepage cutoffs within the foundation (211). Bureau dams have the primary purpose of maintaining reservoirs for irrigation and water supply with incidental hydropower and flood control. On the other hand, dams of the Corps of Engineers are primarily constructed for river improvement which includes flood control, sediment storage, and navigation. Consequently, Corps dams that have limited use of stored water have been constructed with no positive foundation cutoffs.

Rocky Reach Project on the Columbia River was constructed for Public Utility District No. 1 of Chelan County, Washington, as a run-of-the-river hydroelectric project. Extensive grouting of alluvial gravels at the damsite was accomplished in order to reduce piping through the gravels, reduce uplift pressures below the downstream portion of the dam embankment, and to limit seepage to economically acceptable values (60).

Flood control dams serve to reduce flood peaks by temporarily retaining excess flows, and then either allow slow releases in order to not exceed downstream channel capacity, or allow infiltration into pervious foundation strata for recharge of local aquifers (210). Olmos Dam in Texas, owned by the City of San Antonio, is a flood control structure which allows slow releasing of temporarilly stored flood flows. The reservoir area of the dam contains a large recreational park which is not damaged by occasional inundation.

Stability of the upstream slope can be adversely affected by reservoir purpose, and design measures must be implemented to lessen potential for damage. A reservoir pool maintained at relatively constant elevation may cause damage to riprap slope protection due to continued wave action at that constant elevation. Current practice is to provide a very gentle slope, on the order of 1 vertical on 8 horizontal, on the upstream slope at the elevation of the constant pool. Upstream slopes which may be subject to frequent sudden drawdowns of the reservoir pool, such as may occur at flood control projects or hydropower projects, can be zoned to allow quick drainage of near-surface materials. Cottonwood Springs Dam in South Dakota was constructed in 1969 by the Corps of Engineers for flood control, primarily. A pervious drainage blanket eight feet thick was placed on the upstream slope for protection against sudden drawdown conditions. Six inches of bedding, nine inches of spalls and 18 inches of riprap were placed over the pervious drainage blanket (112).

Probable Wave Action

The height of waves in a reservoir depends upon the wind velocity at the water surface, the duration of the wind, the fetch or length of open water, depth of water, and the width of the reservoir. The height of waves may be increased by a decreasing width of the reservoir toward the dam. Upon contact with the embankment slope, the effect of the waves is influenced by the angle of the wave progression relative to the dam, embankment side slope, and texture of the slope surface (204).

The purpose of slope protection is to protect the embankment from erosion which might otherwise be caused by wave attack, surface runoff, or reservoir currents along the slope (203). The upstream slope protection should extend from the crest of the dam to a reasonable distance below minimum reservoir pool. Frequently a thick layer of waste rock is placed below the slope protection to protect the embankment during initial filling. The cost of upstream slope protection frequently is a significant fraction of the total cost of the embankment, particularly for long dams retaining large reservoirs (195).

In addition to damage from wave action, damage can result from ice, as impact damage from large blocks or sheets of ice, or by dislocation and shifting due to riprap becoming frozen into the ice mass.

The most common type of slope protection is dumped riprap, which is a graded rock mass of specified thickness. Increasingly less common is hand-placed riprap, which is of more uniform gradation and therefore more easily degraded by adverse wave conditions. Soil cement is used more often in current practice and is competitive with riprap, provided acquiring the riprap requires a long haul distance. Concrete slope protection for earthfill dams has provided the least satisfactory service due generally to the inherent deficiencies of this type of construction (204). At Copan Dam in Oklahoma, the grade of the upstream slope above the elevation of the conservation pool is 1V on 8H and was originally protected by four feet of impervious material overlain by sod. Substantial gulleying within that portion of the slope eventually required riprap.

Time Available for Construction

As with any engineering project, a profitable return as income or as benefits cannot be made on the initial capital expenditure until the dam is completed and placed into operation. This concept is more appropriately suited, and more easily understood, in regards to small or intermediate earth dams for single purpose reservoirs, especially hydroelectric power projects. Design considerations to hasten construction are more apt to be implemented for these smaller, single-purpose projects.

The overall construction time for major earth dams with multiple-purpose reservoirs is probably not affected to any significant extent by modifying the design in an attempt to shorten the construction time. The legal and environmental considerations of these major projects share equally with technical considerations.

Design considerations which shorten the construction period frequently do not significantly alter the original design, but rather alter the construction process. Measures which may be implemented include construction of the embankment exclusively from borrow sources rather than scheduling construction with required excavation, substituting an upstream slurry trench cutoff in place of an interior earthfill cutoff in order to begin construction of the embankment sooner, and eliminating scheduled layover periods during stage construction by constructing flatter embankment slopes and stability berms.

Types of Earth Dams

Functional Classification

Dams are grouped according to their general purpose, with refinements to the general functional classification based on specific purposes. General functional classifications are storage dams, detention dams, and diversion dams.

Storage Dams. Storage dams provide impoundment of surplus runoff for later use during periods of deficient natural supply. Storage dams are further classified according to the specific purpose of the reservoir, such as water supply, irrigation, or hydroelectric power generation. Design criteria for storage dams require maximum reduction of seepage because of the economic value of the stored water.

Detention Dams. Detention dams are constructed to impound excess flow thereby reducing the adverse effects of floods. Detention dams may temporarily store excess flow in order to significantly reduce the flood peak by lengthening the time of passing and releasing storage at a rate which will not exceed the capacity of the downstream channel. A less common detention dam is of the water-spreading type which impounds the excess flow as long as possible and allows seepage into pervious strata within the reservoir area in order to recharge local water supply aquifers. Detention dams are often constructed as debris dams to trap sediment.

Diversion Dams. Diversion Dams provide head for water conveyance in ditches, canals, and other systems for irrigation, off-channel storage, and municipal and industrial water supply.

Multipurpose Dams. Large water resource projects involving major dams commonly provide multiple purposes which include water supply, irrigation, and flood control.

Hydraulic Design Classification

Dams are also classified according to their hydraulic design as either overflow dams or nonoverflow dams. In more common usage, sections of a dam are classified as overflow or nonoverflow sections. Overflow dams or sections are designed for discharge over their crests and must be constructed of nonerodable material such as concrete, masonry, or steel. Nonoverflow dams or sections are not designed to be overtopped and extends the choice of construction materials to include earthfill and rockfill.

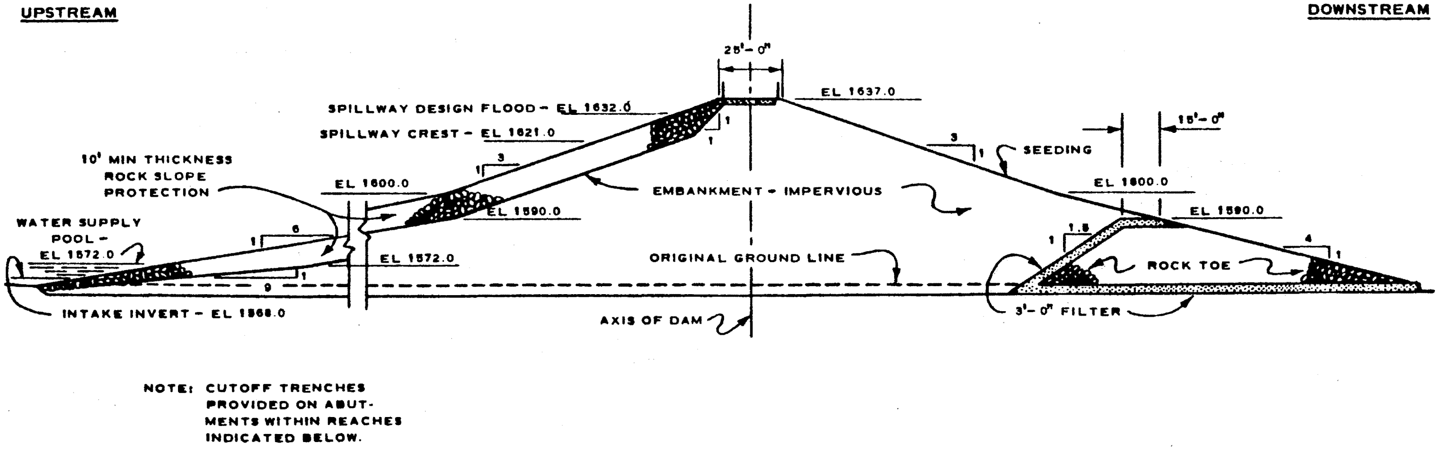
Earthfill Dams

Earthfill dams are the most common type of dam, principally because of their utilization of natural materials for construction (210). Futhermore, the foundation requirements for earthfill dams are less stringent than for other types, to the extent that earthfill dams have been constructed on very poor foundations (214). Although the earthfill classification is not restricted to rolled-earthfill, the development and economic impact of modern excavating, hauling, and compacting equipment to construct rolled-earthfill dams has virtually eliminated semihydraulic-fill and hydraulic-fill construction of earthfill dams. Henceforth within this text, the term earthfill refers only to rolled-earthfill.

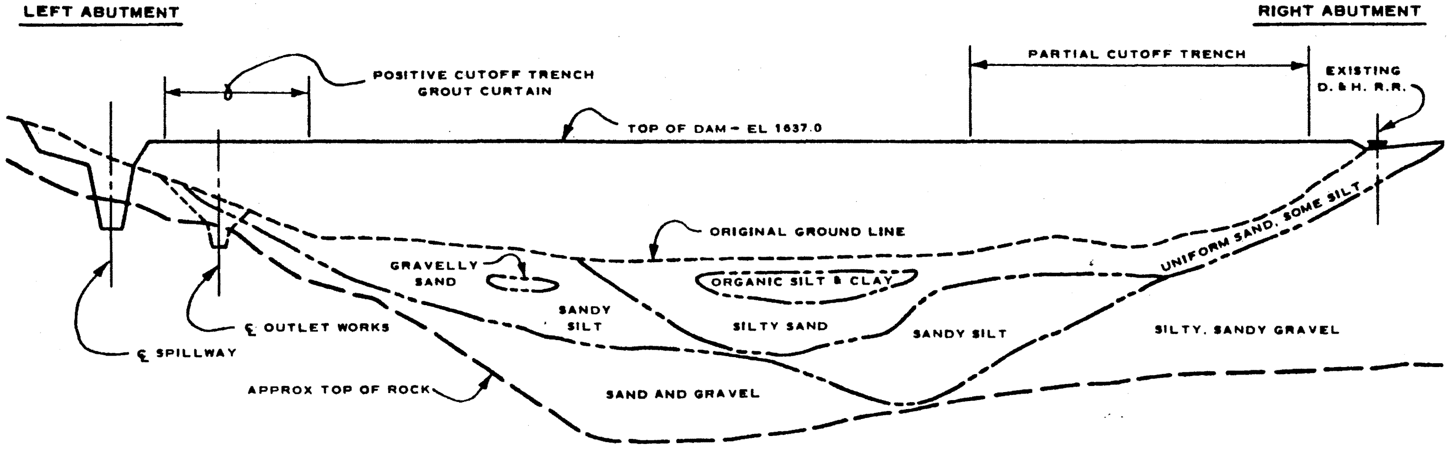
Homogeneous Dams. Available embankment materials which are predominantly of one soil type or are too variable to separate for placement in specific zones result in the construction of a homogeneous earthfill dam. Small dams are almost universally constructed as homogeneous embankments because of a limited construction area which is much too narrow for the surveying and control complexities of zoned embankments. These ubiquitous small dams impound reservoirs from farm ponds to municipal water supply. Infrequently, homogeneous embankments are constructed to moderate heights such as the 159-foot high Whitney Dam in Texas.

Homogeneous dams should be provided with internal drains in order to: (1) reduce the uplift pressures below the downstream portion of the embankment and hence increase slope stability, and (2) reduce the piping potential of the embankment materials by controlling the seepage discharge (57). Exceptions would be very small dams which, if failed, would not present an economic loss through loss of the reservoir or through downstream damage. The simplest drain is a rock toe, usually surrounded by a graded filter to prevent piping of embankment materials into the drain. Stillwater Dam in Pennsylvania was provided with such a rock toe and is shown in Figure 5.

Horizontal filter blankets have been used in the downstream portion of homogeneous dams of low to moderate height as a means to intercept and convey seepage. Horizontal filter blankets have also been used to increase the rate of consolidation of impervious foundation soils. At Hulah Dam in Oklahoma the horizontal filter blanket was placed to within 50 feet of the upstream embankment toe (166). At Denison Dam in Texas a gravel blanket was placed to the upstream toe of the embankment in a segment of the foundation. Seepage reduction was achieved somewhat by sheet piling (149). The Bureau of Reclamation recommends placement of the horizontal filter from the downstream toe to a distance of the height of the dam plus five feet, which places the



TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (185)

Figure 5. Stillwater Dam, Lackawanna River, Pennsylvania.

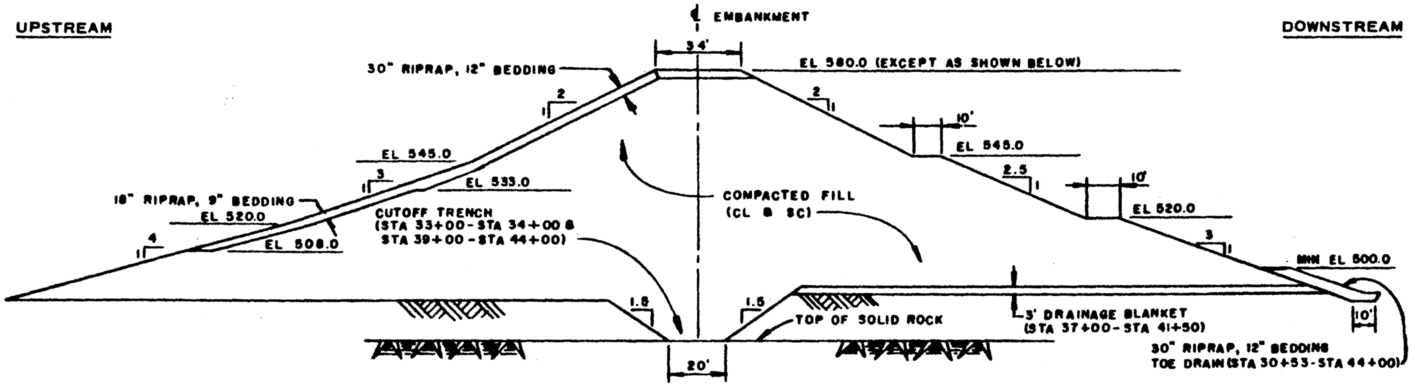
upstream limit of the filter at the downstream edge of a minimum required core thickness (210). A disadvantage of horizontal filter blankets, related to seepage control, results from the inevitable stratification of rolled-earthfill and the corresponding greater horizontal permeability (57). A disadvantage, related to construction pore pressures within the embankment is the long seepage path through the embankment to obtain dissipation (74). A horizontal filter blanket was placed at Whitney Dam in Texas as shown in Figure 6.

The disadvantages of horizontal filter blankets are overcome by inclined or vertical chimney drains which intercept seepage within the stratifications and provide substantially shorter seepage paths for consolidation of the earthfill. An inclined drain was provided at Navarro Mills Dam in Texas and a vertical drain was provided at Hopkinton Dam in New Hampshire. The typical embankment sections of these two dams are shown in Figures 7 and 8, respectively.

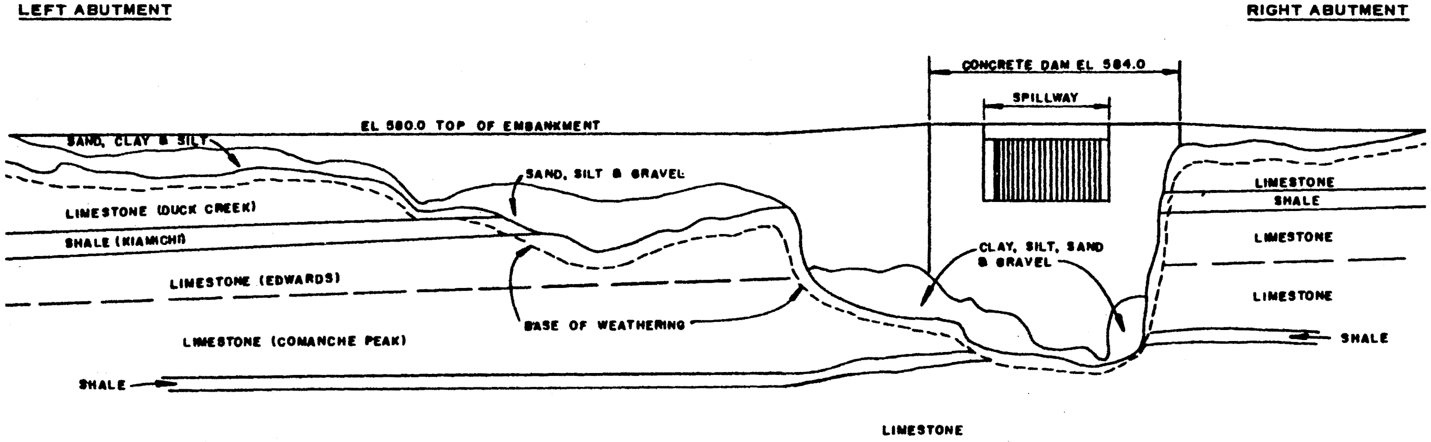
North Springfield Dam in Vermont was constructed of gravelly silty sand as a homogeneous embankment, as shown in Figure 9. The foundation is silty sand. A horizontal filter blanket was not used below the embankment because of the pervious nature of the dam and its foundation. Horizontal seepage through the stratified embankment is intercepted by a downstream inclined filter and discharged into a rockfill berm (177).

The embankment of Northfield Brook Dam in Connecticut is essentially homogeneous, but contains an interior zone of random material which was not deemed suitable as impervious material (176). The typical embankment section is shown in Figure 10.

During construction of some homogeneous dams, attempts have been



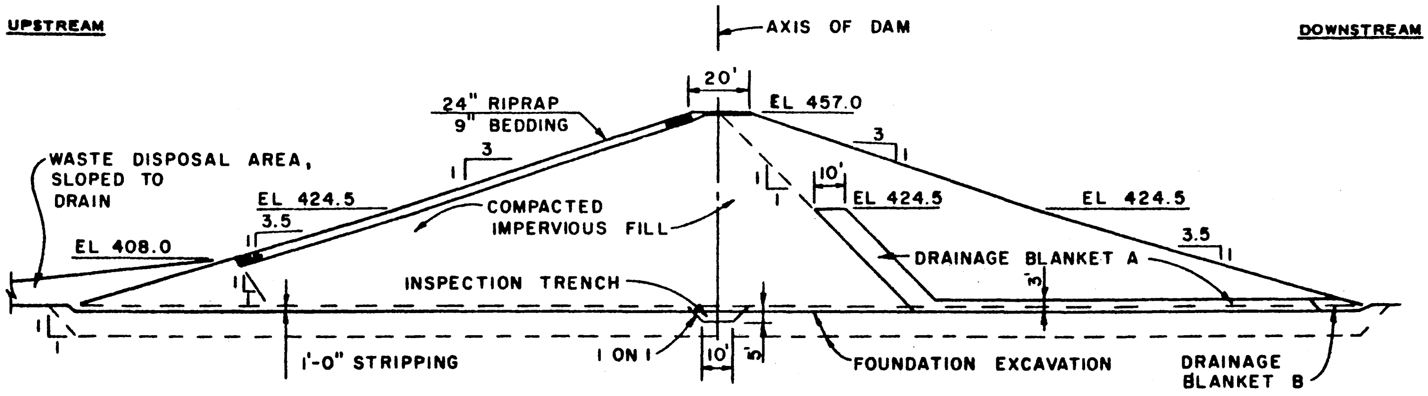
TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (193)

Figure 6. Whitney Dam, Brazos River, Texas.



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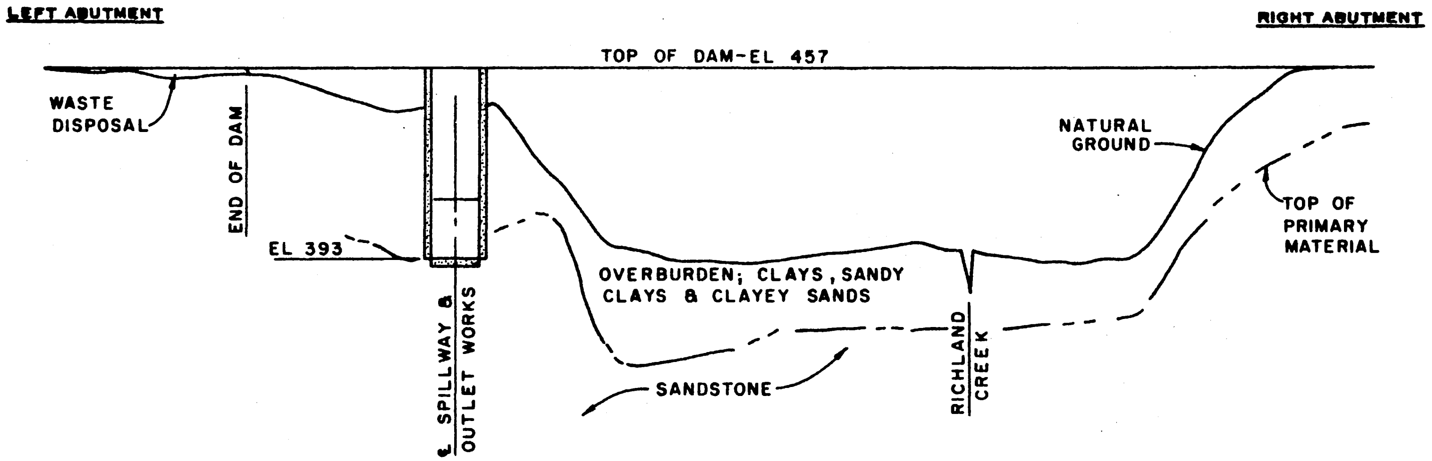
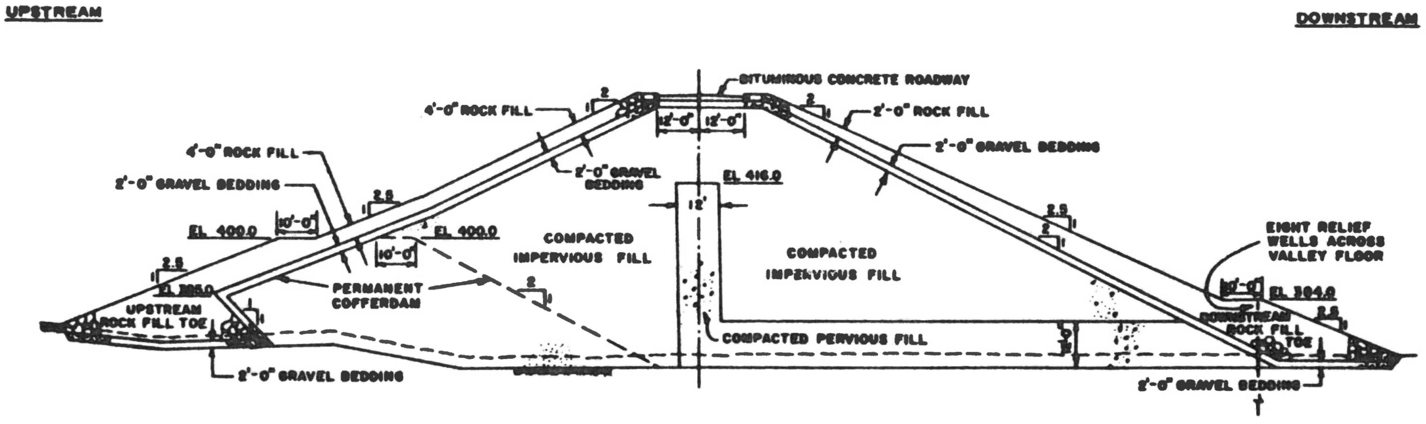


VALLEY CROSS SECTION

Source: (174)

Figure 7. Navarro Mills Dam, Richland Creek, Texas.



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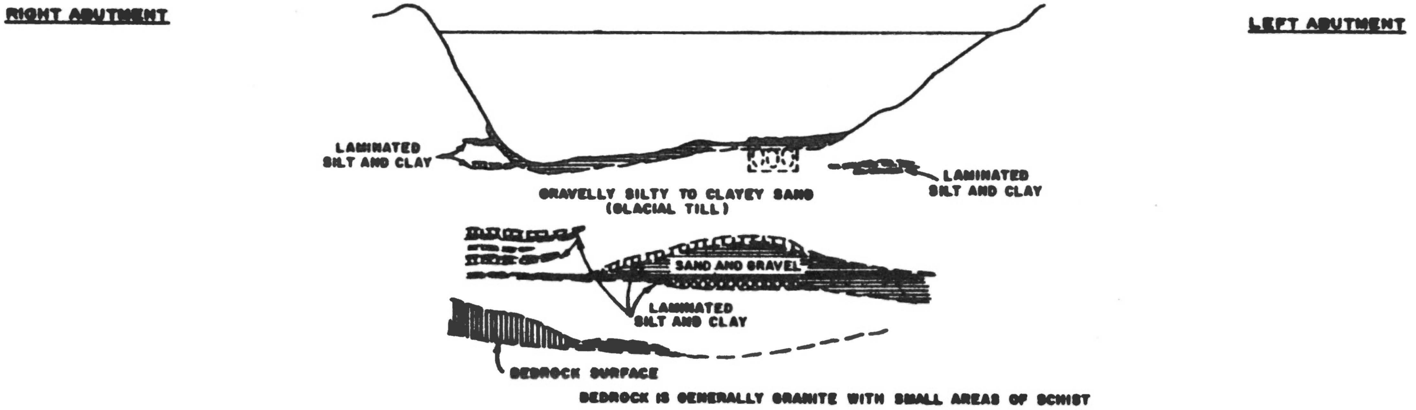
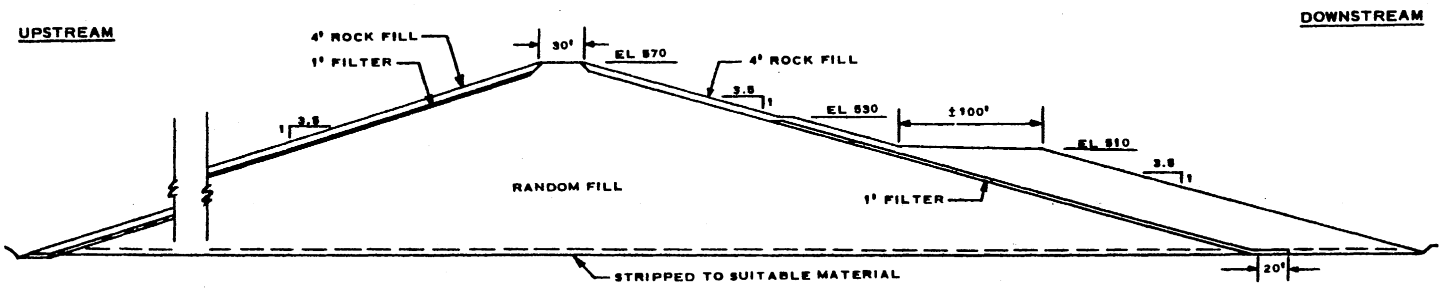


VALLEY CROSS SECTION

Source: (164)

Figure 8. Hopkinton Dam, Contoocock River, New Hampshire.



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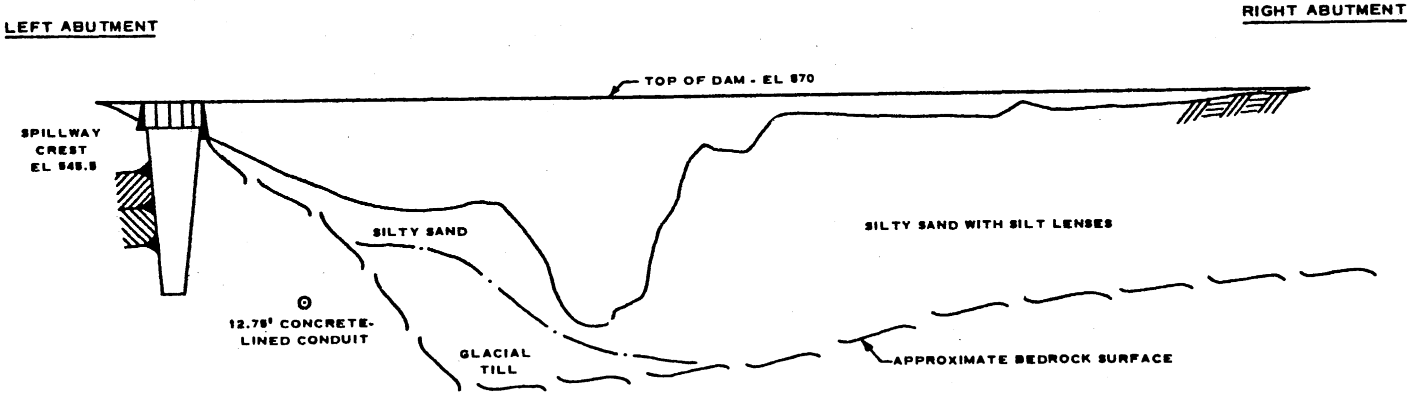
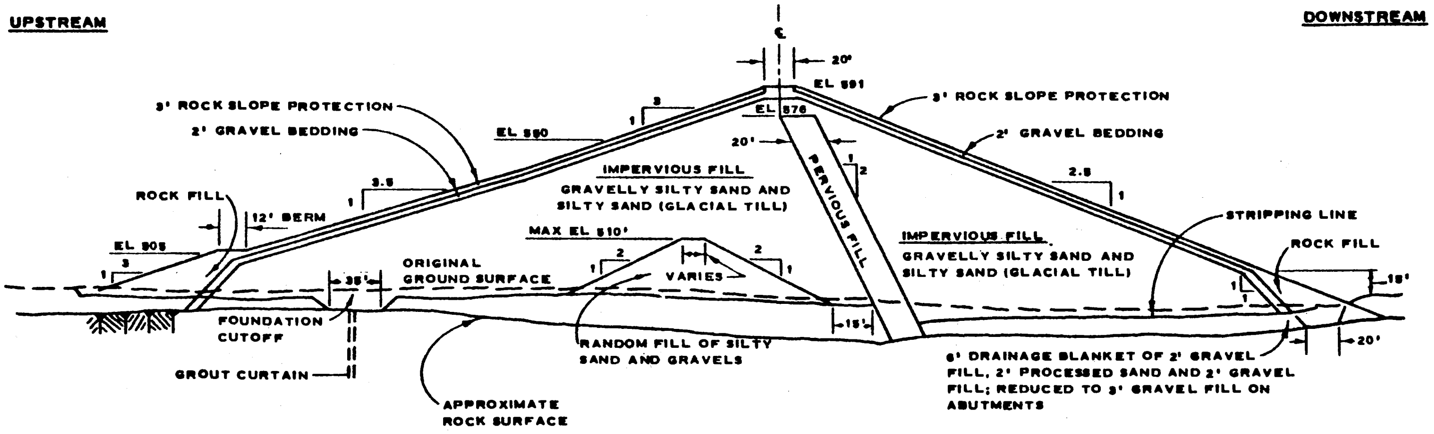


VALLEY CROSS SECTION

Source: (177)

Figure 9. North Springfield Dam, Black River, Vermont.



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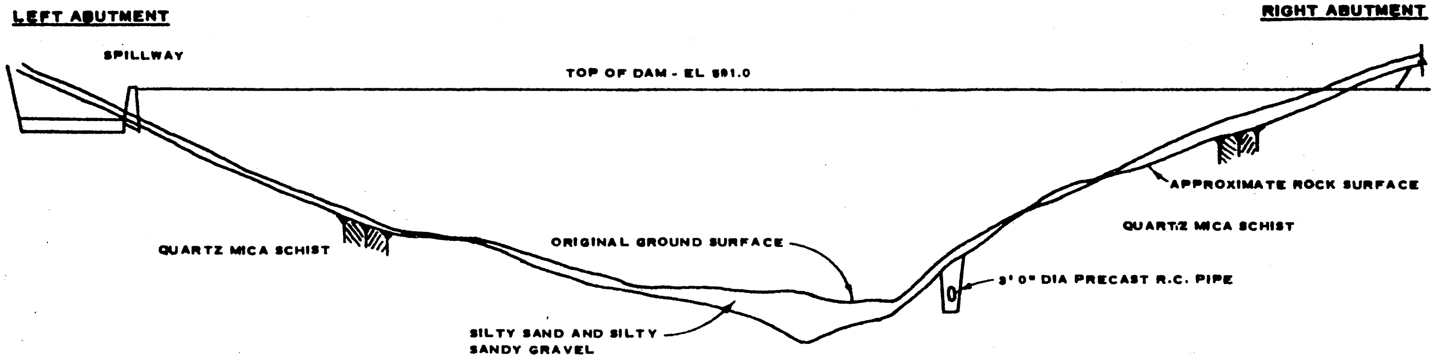


VALLEY CROSS SECTION

Source: (176)

Figure 10. Northfield Brook Dam, Northfield Brook, Connecticut.

made to place the more pervious materials toward the outer edges of the embankment fill and the more impervious materials in the center. This tends to some extent to create the effect of a zoned embankment. However, because of an absence of definitive construction controls and appropriate shear strength data these dams should be considered homogeneous dams for analyses. Some zoned embankments are constructed from the same borrow material, but because of processing specifications for the different zones the permeabilities, shear strengths, and stress-strain characteristics are distinctly different. These embankments are rightly considered zoned embankments, not homogeneous embankments.

Thin Core Dams. Earthfill dams with thin impervious cores are usually constructed when local borrow sources do not provide ample quantities of impervious material. Also, thin core dams are occasionally constructed because of economic and scheduling advantages, in spite of ample impervious materials.

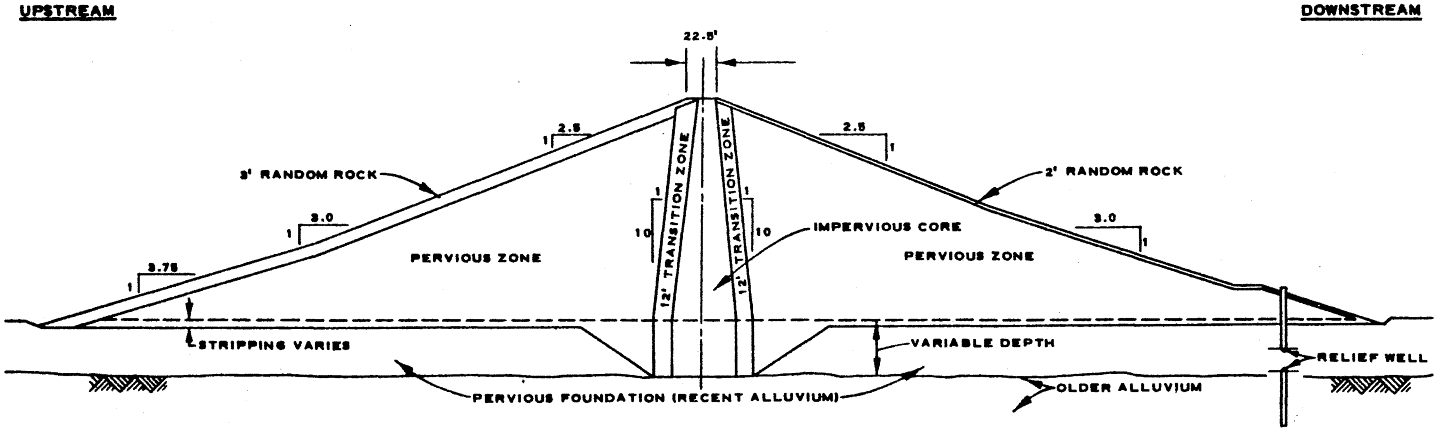
Pervious materials, herein referring to relatively clean sands and gravels, provide higher shear strengths than can be expected from impervious or random material. Consequently, side slopes of pervious embankments, or embankment shells, may be constructed steeper than slopes of impervious or random embankments, and with steeper side slopes the embankment involves less volume of earthfill and less construction cost. Also, unit construction costs of placement of pervious materials is somewhat less than unit costs of impervious because of less required processing to achieve suitable fill.

In areas of moderate to high rainfall, construction progress of large zones of impervious material can be subtantially slowed due to wet borrow material and increased difficulty during compaction. Construction of free-draining pervious materials are not unduly hampered by inclement weather. Dams designed with upstream sloping cores allow completion of the downstream pervious zone prior to construction of the impervious core. This construction technique is ideally suited to areas of limited dry weather. Construction of the downstream pervious zone can also be accomplished simultaneously with construction of a cutoff trench and grouting of foundation rock.

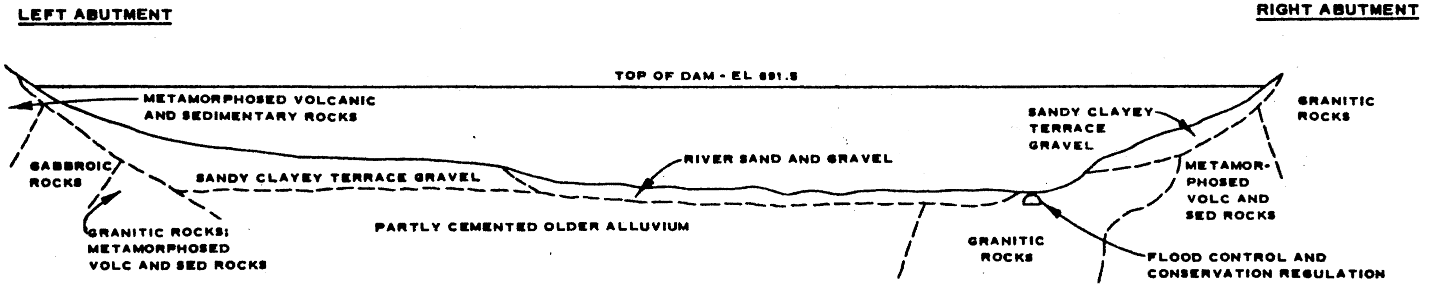
The width of impervious core is governed by the width required for seepage control and piping resistance. The minimum width of the core has been established as 30 percent of the hydraulic head at the respective elevation (57). For construction convenience, the practical minimum width at the top of the core should be 10 feet (128).

A good example of a thin core dam is provided by Success Dam in California, completed in 1961 by the Corps of Engineers and shown in Figure 11. The dam has a maximum height of 142 feet above the streambed and was constructed with a very thin central core flanked by pervious shells. The material for the impervious core was obtained from an upstream borrow area with an average haul distance of two and one-half miles. The impervious material is predominantly sandy clays and clayey sands. The materials for the pervious shells, consisting of sands and gravels, were obtained from recent alluvial deposits upstream and downstream of the damsite with an average haul distance of one-half mile (94).

An example of a sloping core dam is provided by West Hill Dam in Massachusetts, also completed in 1961 by the Corps of Engineers and



TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

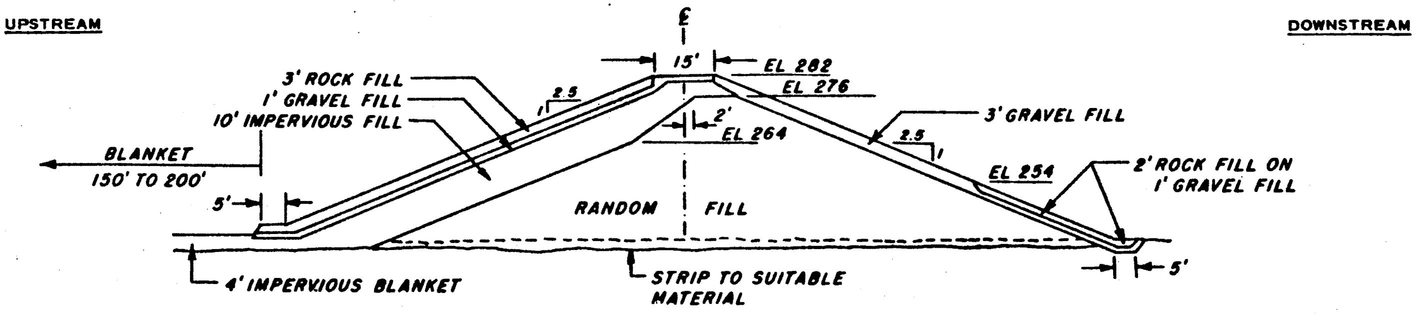
Source: (94)

Figure 11. Success Dam, Tule River, California.

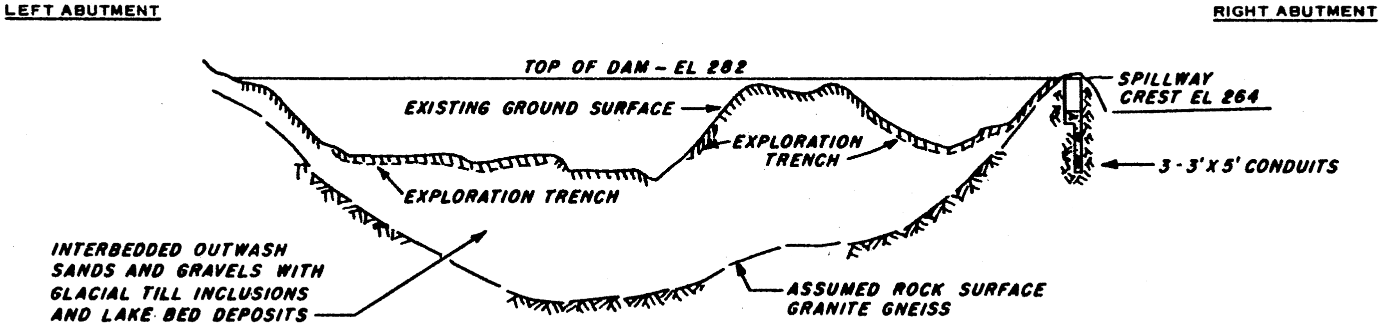
shown in Figure 12. The embankment has a maximum height of 48 feet with a 10-foot thick impervious core placed on the upstream slope of the pervious zone. The impervious material consisted of gravelly silty sands from glacial till and the pervious zone was constructed of glacial outwash and is predominantly gravelly sand (190).

Hills Creek Dam in Oregon, also completed in 1961 by the Corps of Engineers, is a thin-core, earthfill dam with a maximum height of 325 feet. The typical embankment section of Hills Creek Dam is shown in Figure 13. The embankment consists of a central impervious core of clayey and silty gravel with gravel and rock shells. The downstream shell consists of well-graded sandy gravel; the upstream shell has a lower random rock section with an upper interior gravel section underlying an exterior free-draining rock section. Random rock was obtained from required excavation. Gravel was excavated from reservoir floodplain deposits, and impervious material was borrowed from a downstream source about one mile from the dam (101). Hills Creek Dam illustrates the steeper side slopes possible with outer shells constructed of gravel.

Zoned Embankment Dams A thin core dam is a specialized zoned embankment dam, and, perhaps, a homogeneous dam is also a specialized zoned embankment dam. However, the term zoned embankment dam conjures up the image of numerous thin zones across an embankment section, each serving a specific purpose and each transitioning into the next. This type of section is rare for earthfill dams because of the relatively close gradation of soil material for embankments. That type of section is more common for rockfill dams which grade from boulder-size rock at



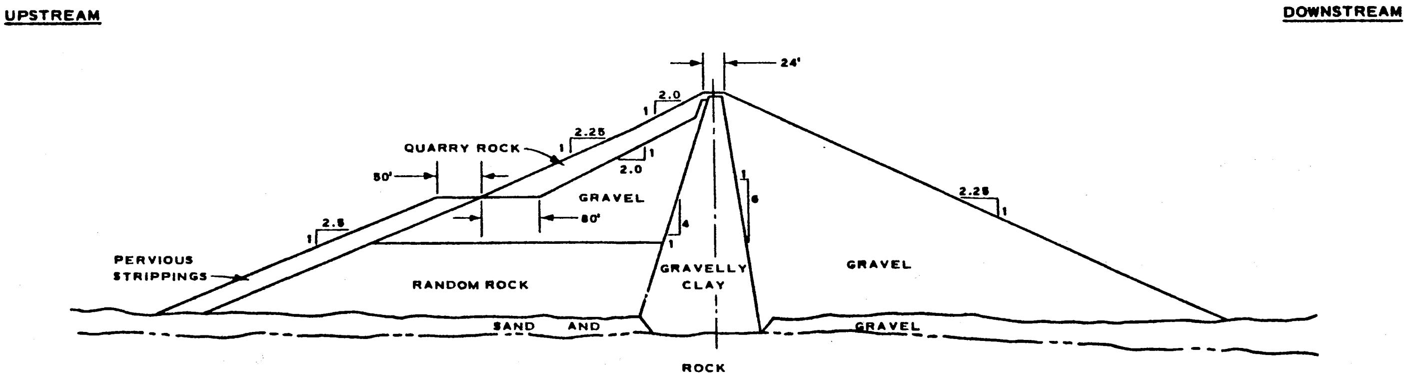
TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (190)

Figure 12. West Hill Dam, West River, Massachusetts.



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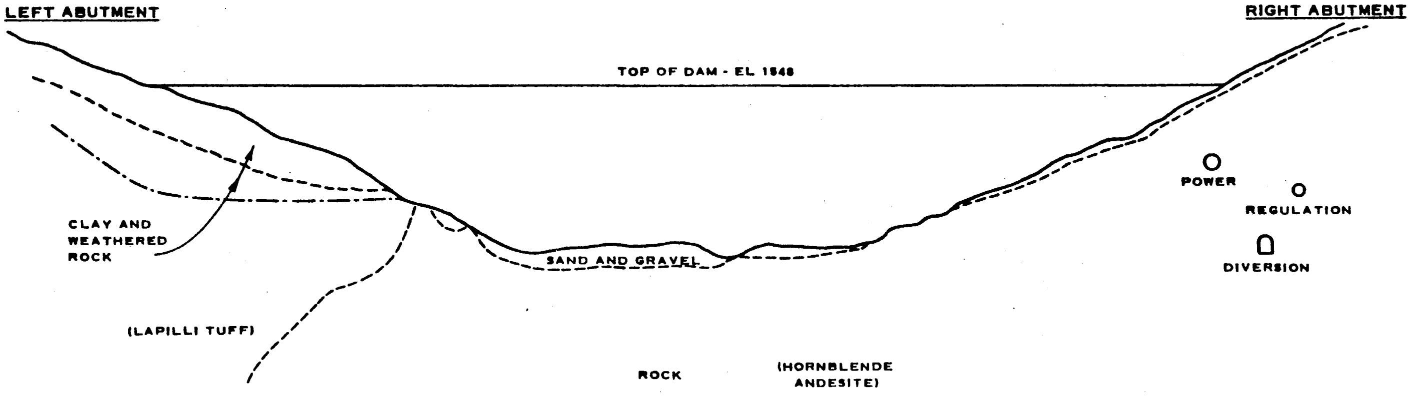


VALLEY CROSS SECTION

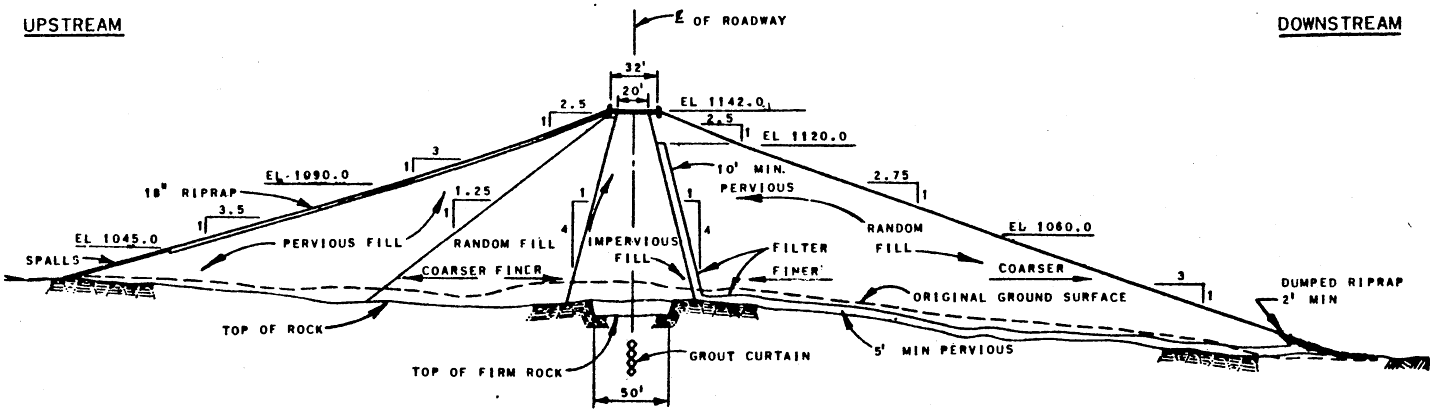
Source: (101)

Figure 13. Hills Creek Dam, Middle Fork Willamette River, Oregon.

the outer edges of the embankment to fine-grained impervious soils at the core.

Some good examples of zoned embankments are provided by Beaver Dam in Arkansas, Coralville Dam in Iowa, and Tenkiller Ferry Dam in Oklahoma, shown in Figure 14, Figure 15, and Figure 16, respectively. At Beaver Dam, random materials of clay with 50 percent rock content were placed with coarser materials toward the outer edges of the random zones and finer materials toward the impervious core. This selective placement within the random zones created a transition effect between the upstream pervious zone of gravels and the impervious silty clay core (139). At Coralville Dam, random zones of sandy clay provided transition between the pervious outer zones of sand and the impervious core of loess (76). At Tenkiller Ferry Dam, a central impervious core of clayey silt is flanked by semi-impervious shells of sandy gravel. The more pervious sandy gravel was selectively excavated for placement in the far downstream semi-impervious shell (186).

The embankment of Elk City Dam in Kansas, presented in Figure 17, is typical of zoned embankment dams which are constructed of a common borrow material. The zoning of the dam is reflective of the required fill processing, not of the type of material. The central impervious core was compacted at slightly higher moisture content and with more stringent control of material properties, in order to provide a more plastic core (153). A similar highly plastic core was obtained at Canyon Dam in Texas, shown in Figure 18, but as much by type of material as by compaction specifications. The central impervious core was constructed of high plasticity clays at a higher moisture content than the materials of the outer zones. The outer impervious shells



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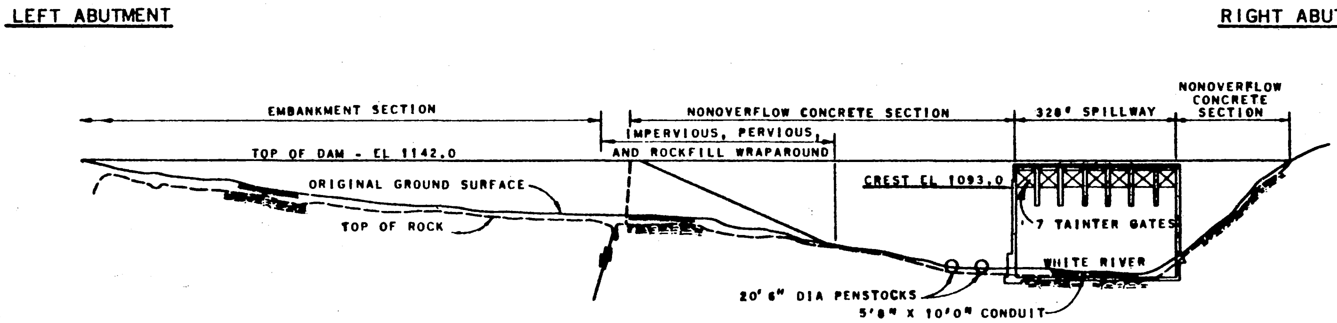
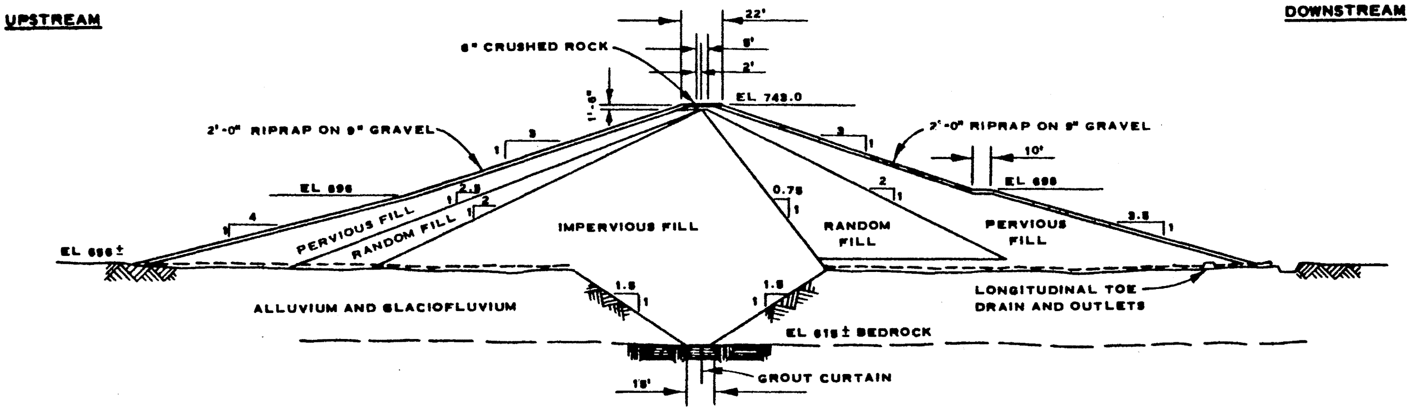


VALLEY CROSS SECTION

Source: (139)

Figure 14. Beaver Dam, White River, Arkansas.



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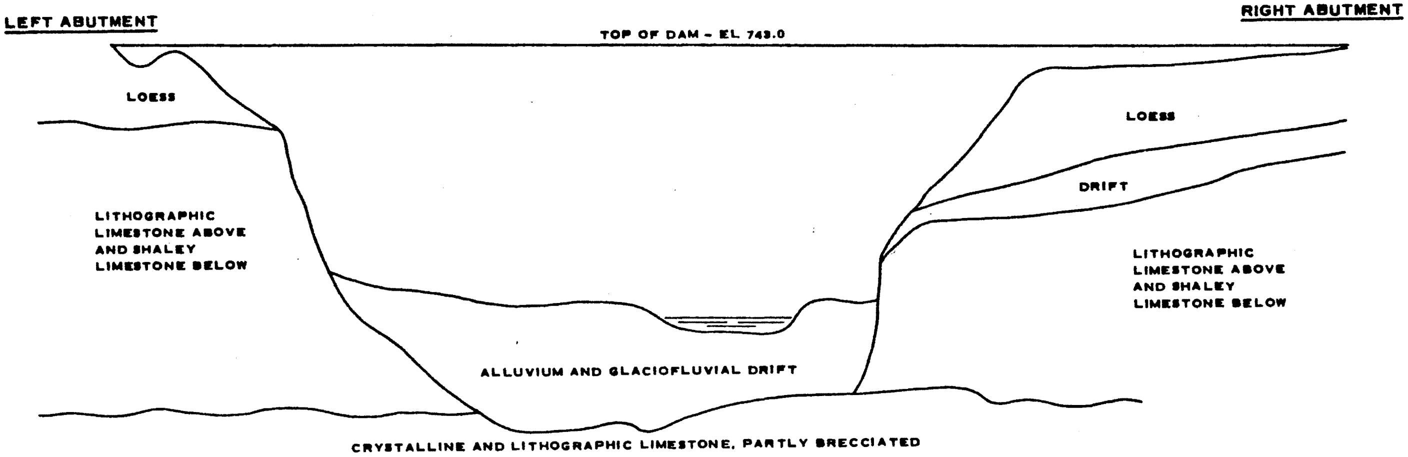
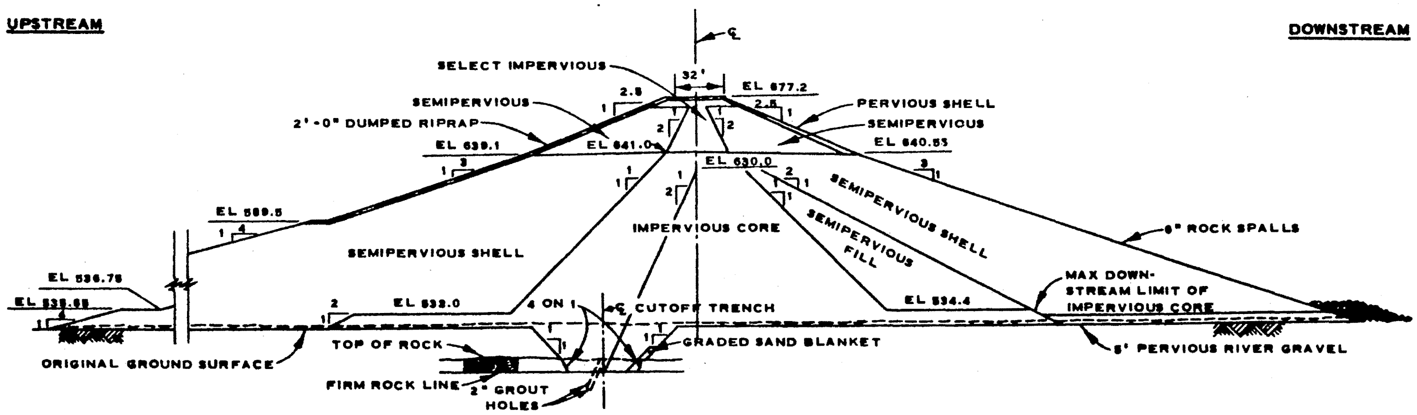


VALLEY CROSS SECTION

Source: (76)

Figure 15. Coralville Dam, Iowa River, Iowa.



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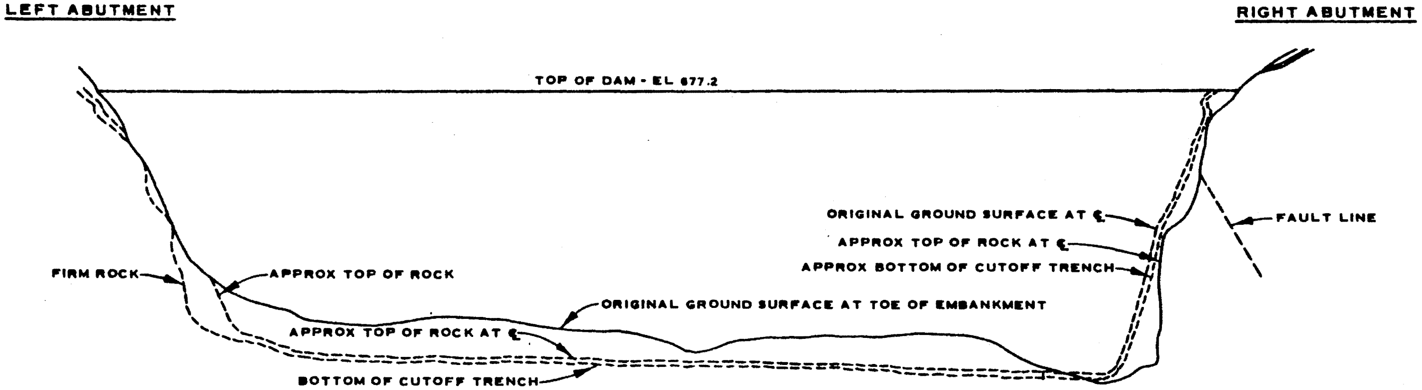
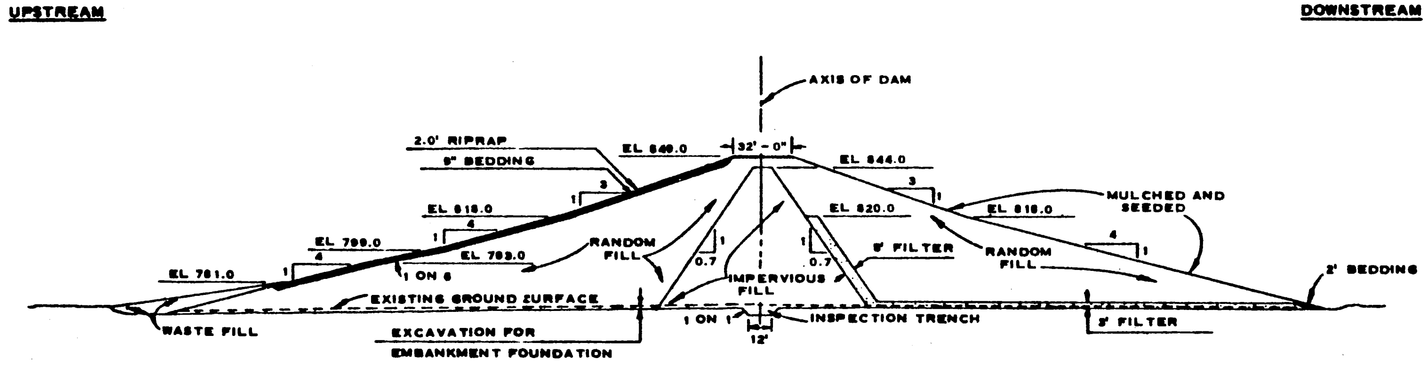


VALLEY CROSS SECTION

Source: (186)

Figure 16. Tenkiller Ferry Dam, Illinois River, Oklahoma.



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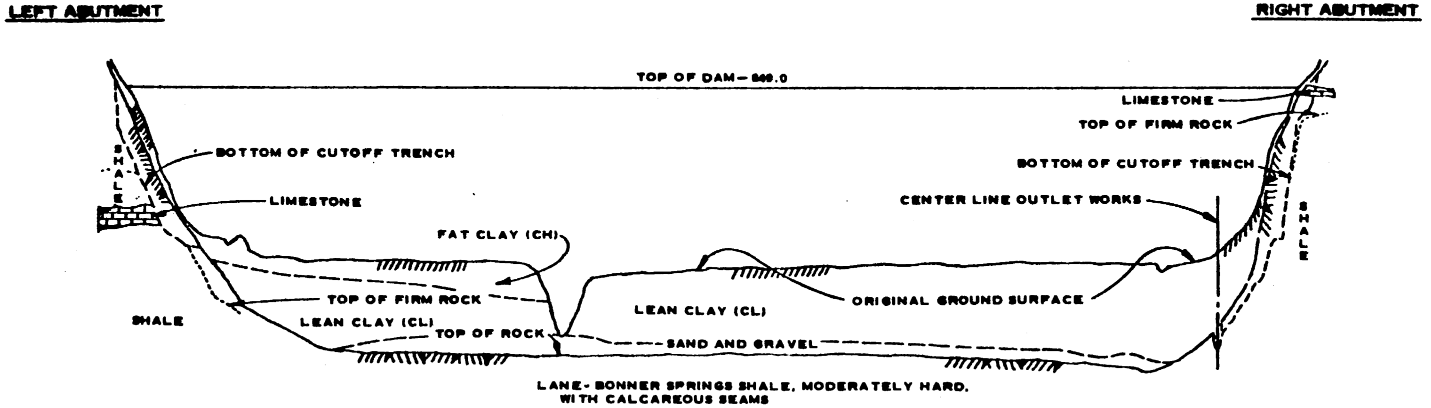
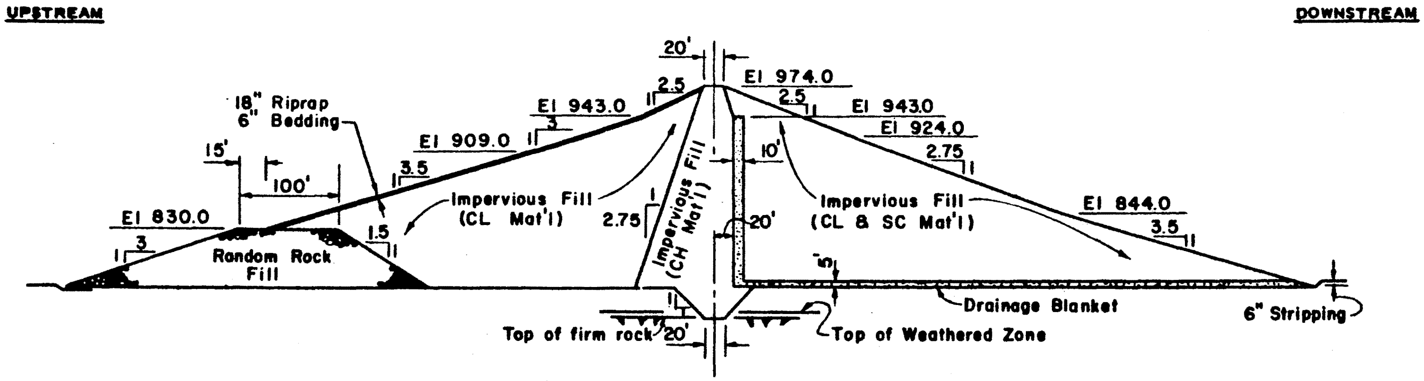


VALLEY CROSS SECTION

Source: (153)

Figure 17. Elk City Dam, Elk River, Kansas.



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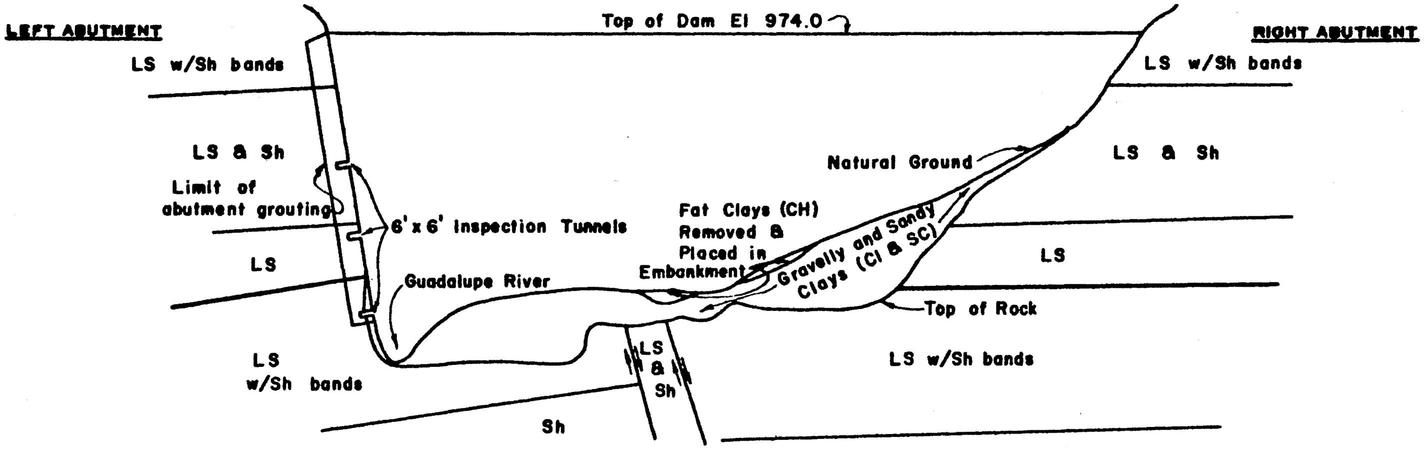


VALLEY CROSS SECTION

Source: (98)

Figure 18. Canyon Dam, Guadalupe River, Texas.

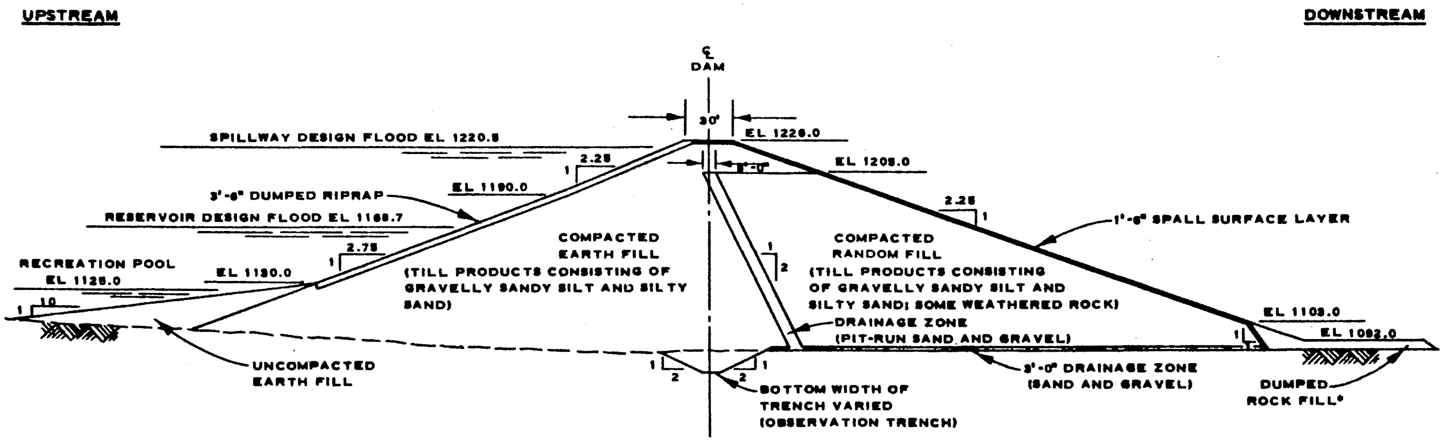
were constructed of low plasticity clays with selective placement of moderately plastic clays adjacent to the central core to provide some degree of transition (98).

Zoned embankments with an upstream impervious section and a downstream random section are common for dams of intermediate height. These dams are quite often constructed from a common borrow source with more selective placement and compaction control in the impervious zone. Two examples are provided by Prompton Dam in Pennsylvania and Diamond A Dam in New Mexico, shown in Figures 19 and 20, respectively.

Rockfill Dams

Originally the term rockfill dam referred only to dams constructed entirely of rock with an impervious membrane along the upstream slope. The Eleventh Congress of the International Committee on Large Dams (ICOLD) in 1940 defined a rockfill dam as a dam which utilizes rockfill as a structural element (18). This definition was adopted by the American Society of Civil Engineers (ASCE) in 1958. A rockfill dam within this text is as defined by the Corps of Engineers as any earth dam constructed predominantly of rockfill materials (128).

Rockfill dams can prove economical when any of the following conditions exist: (1) large quantities of rock are readily available from nearby borrow sources or from required excavation; (2) earthfill is difficult to obtain or requires extensive processing; (3) short construction seasons prevail; (4) excessively wet climate; or (5) the dam is to be raised at a later date (210). Other factors which favor rockfill dams are the ability to place rockfill in freezing climates and the ability to conduct foundation grouting with simultaneous



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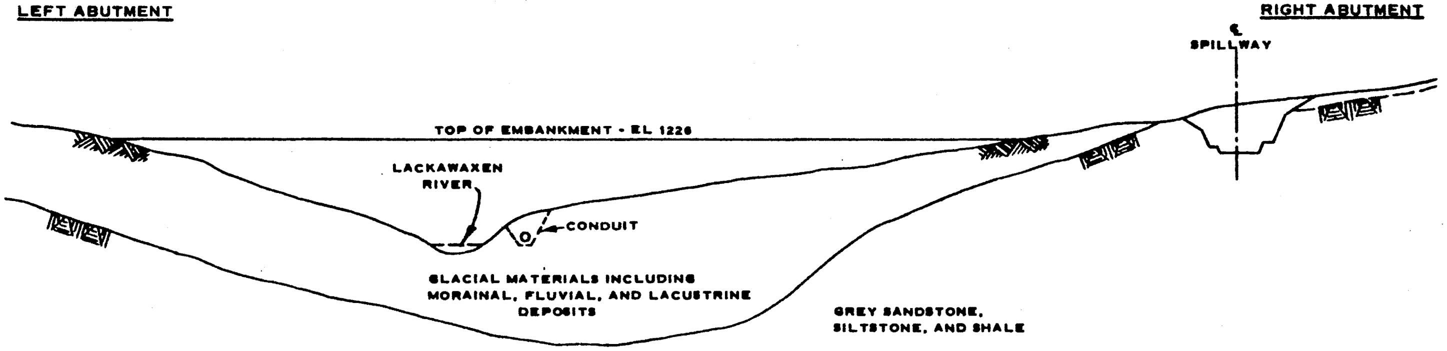
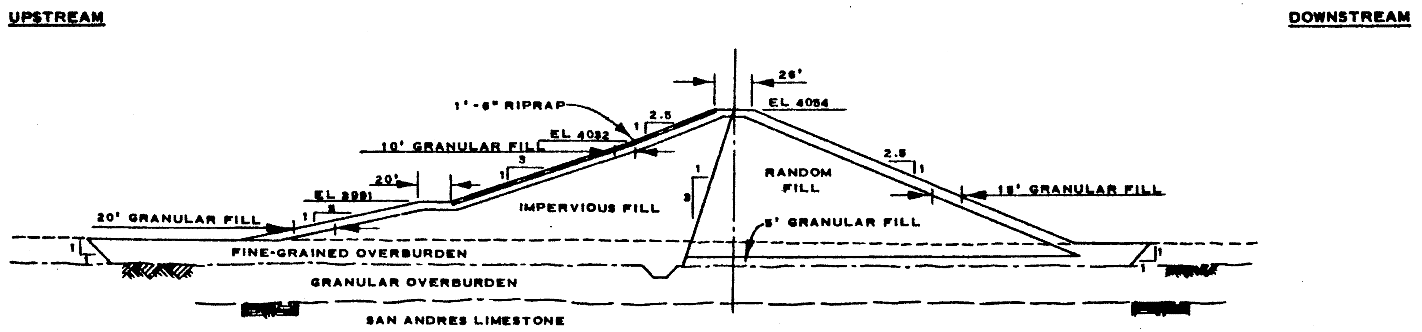


VALLEY CROSS SECTION

Source: (180)

Figure 19. Prompton Dam, Lackawaxen River, Pennsylvania.



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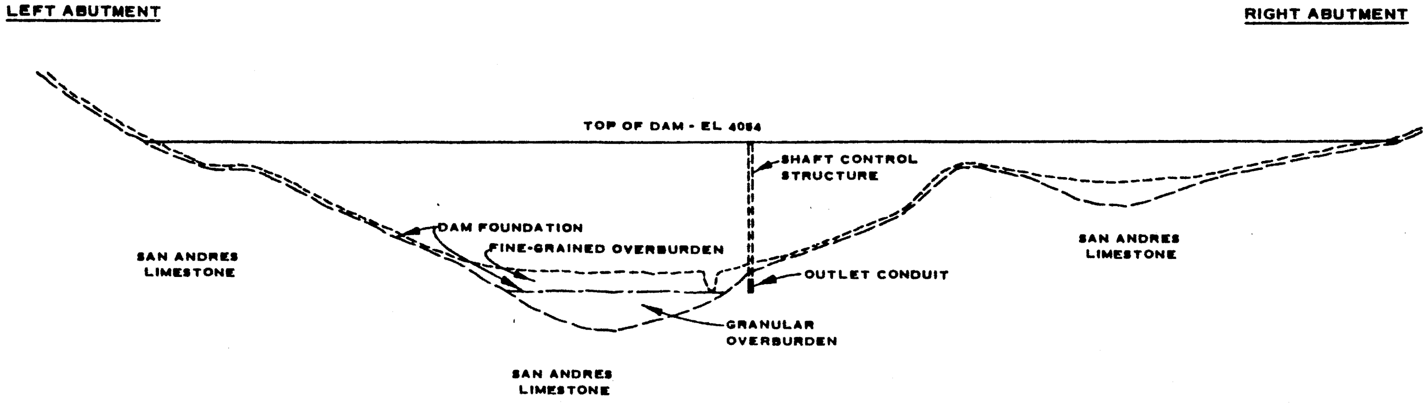


VALLEY CROSS SECTION

Source: (150)

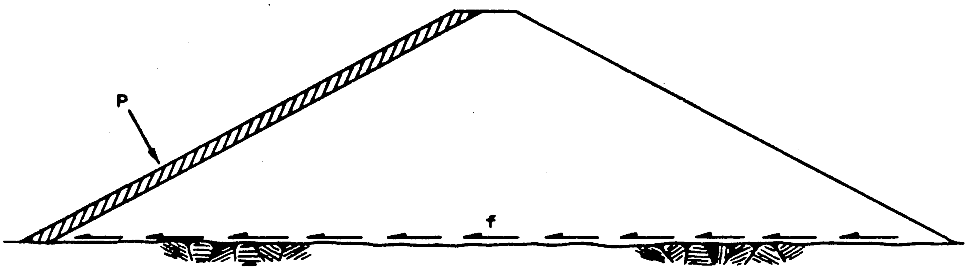
Figure 20. Diamond A Dam, Rio Hondo, New Mexico.

placement of rockfill for sloping core and decked dams.

Rockfill dams may be divided into four groups depending on the location of the impervious core or membrane, as follows: (1) central core, (2) sloping core, (3) upstream membrane, or decked, and (4) earth-rock. The effect of the location of the impervious core or membrane on stability of the rockfill embankment can readily be seen in Figure 21. The farther upstream the core or membrane is located, the greater the resistance to sliding of the embankment (210).

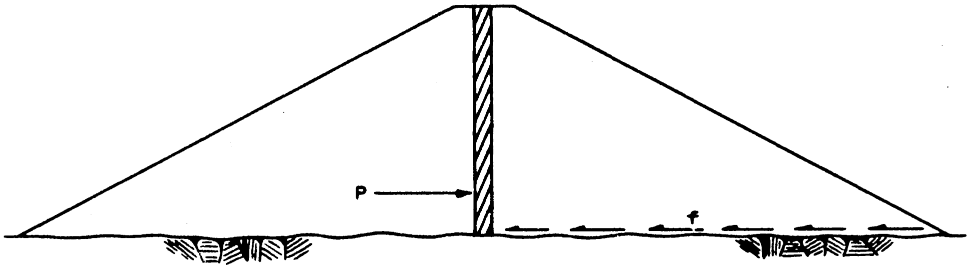
A great variety of rock types has been used in the construction of rockfill dams. The types of rock used range from hard, durable granite and basalt to weaker materials such as weathered conglomerate and weathered shale. The 445-foot rockfill embankment of Cougar Dam in Oregon, Figure 22, was constructed of basalt and has side slopes of 1V on 1.8H which is slightly steeper than normally expected from the best rockfill materials. The 154-foot high embankment of Tionesta Dam in Pennsylvania, Figure 23, was constructed in 1941 using a mixture of overburden soils and shale, ratio of 1 to 1, in the random zones. the outer pervious fill and rockfill zones of Tionesta Dam consist of sandstone. The shale was mixed with overburden soil for use in the embankment rather than used alone because of the lack of experience at the time in compacting weathered shale (208). The 210-foot high embankment of East Fork Dam in Ohio was constructed using interbedded shale and limestone in random rock and rock zones. The interbedded shale and limestone was excavated by blasting, placed in 8-inch loose lifts, and compacted by both a tamping roller and a rubber-tired roller (26).

The foundation requirements for a rockfill dam are less severe



(B) UPSTREAM MEMBRANE

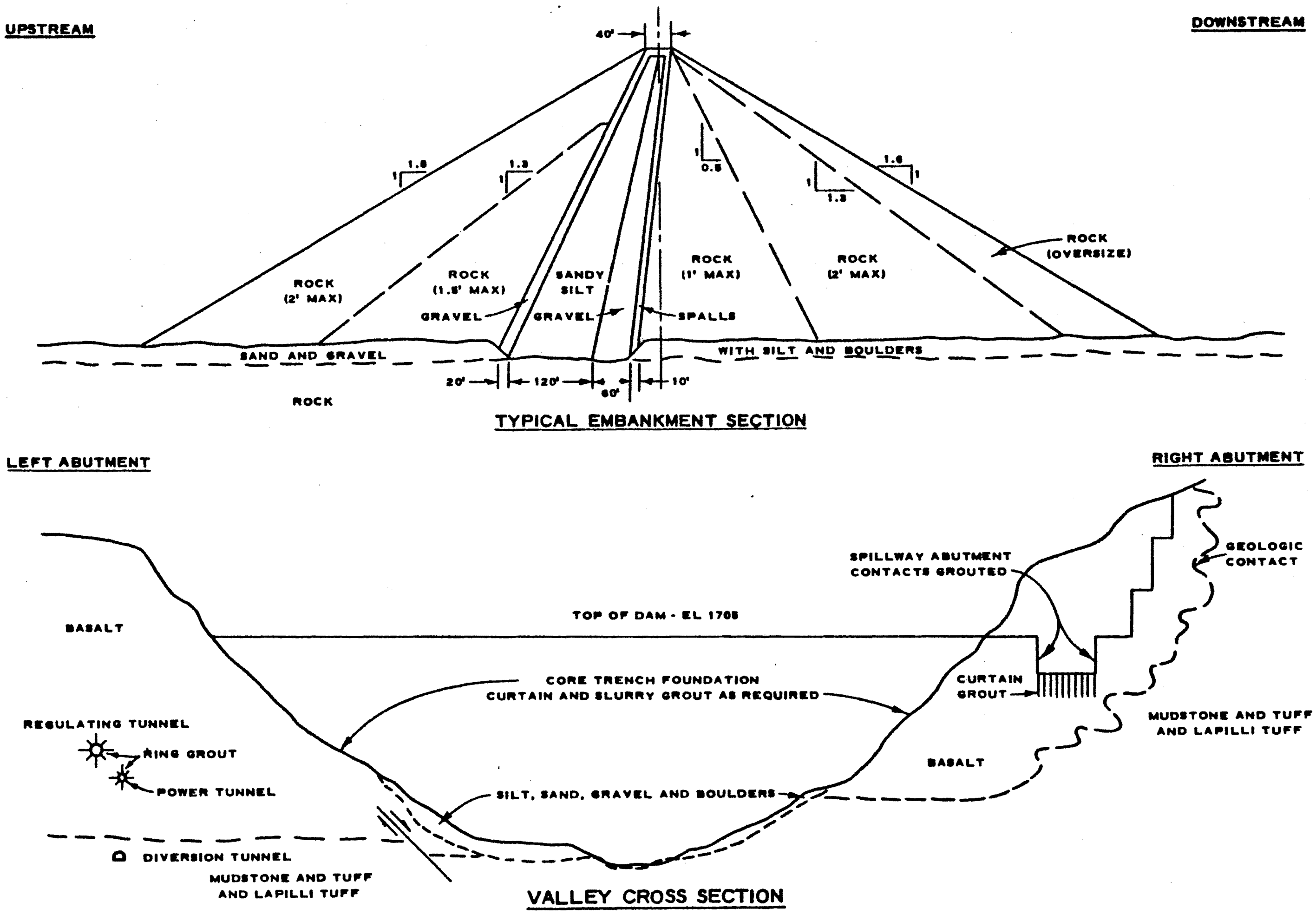
P = Resultant water force
f = Friction forces resisting sliding



(A) CENTRAL MEMBRANE

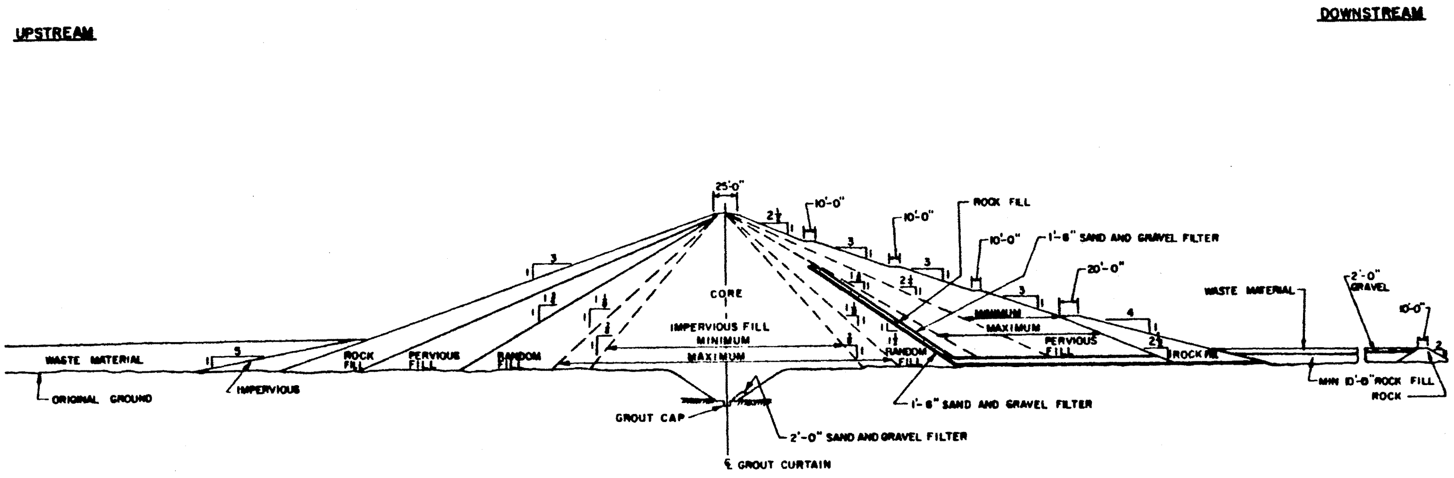
Source: (210)

Figure 21. Effect of Membrane Location on Resistance of Embankment to Sliding.

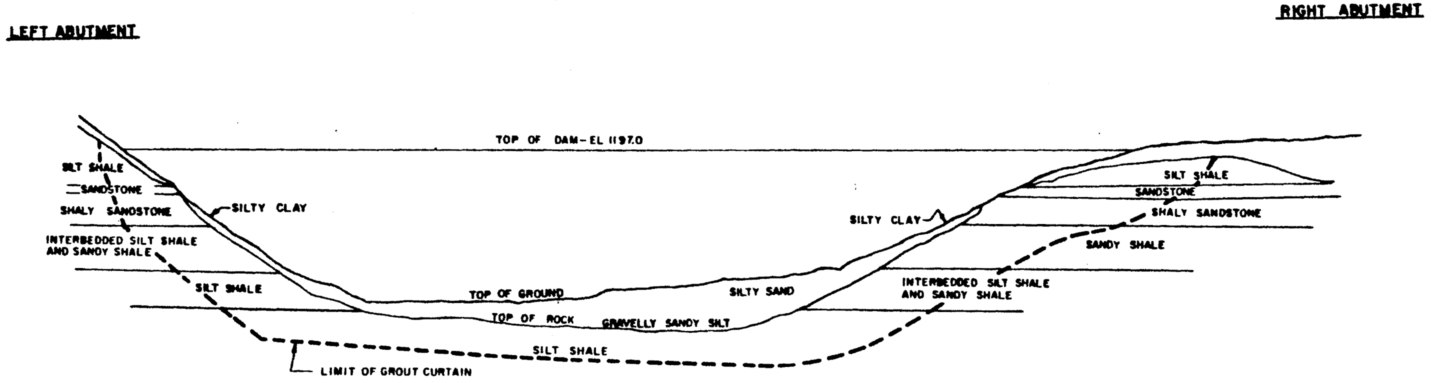


Source: (92)

Figure 22. Cougar Dam, South Fork McKenzie River, Oregon.



TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (187)

Figure 23. Tionesta Dam, Tionesta Creek, Pennsylvania.

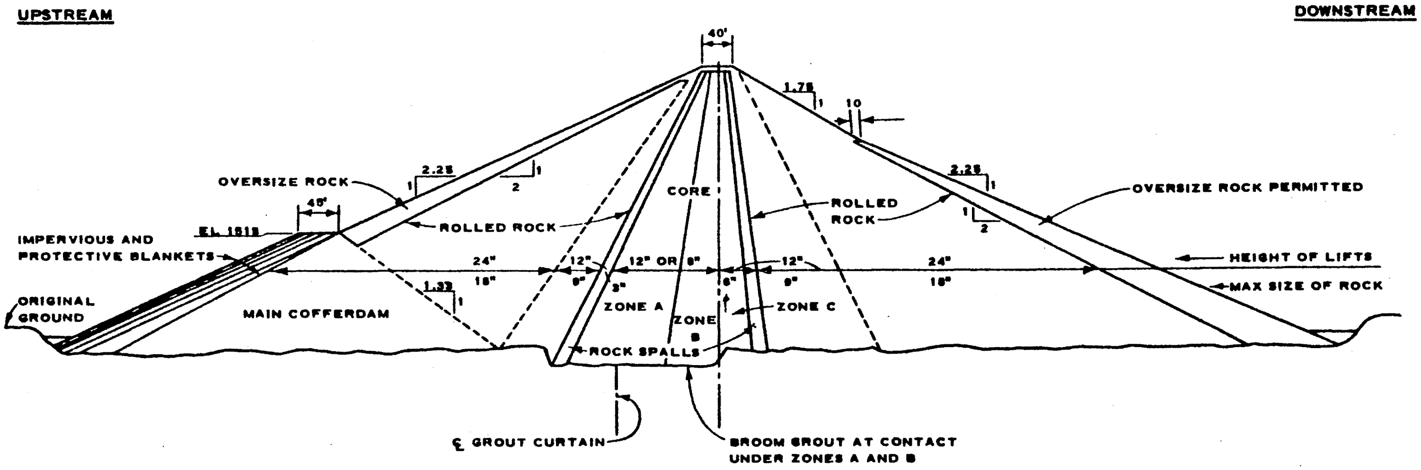
than for a concrete gravity dam, but are more severe than for an earthfill dam (210). Massive, steep-sloped rockfill dams, such as Cougar Dam, require construction directly on sound foundation rock. However, as the rockfill, because of natural decomposition and thorough construction processing, more closely resembles soil, the foundation requirements more closely resemble the requirements for earthfill dams.

Early rockfill dams used upstream membranes exclusively and were constructed with steep slopes, usually 1V on 0.5 to 0.75 H, to minimize the volume of rockfill. Since these slopes were considerably steeper than the natural angle of repose of dumped rock, the slopes were stabilized by thick zones of crane-placed, dry rubble masonry. In current practice rockfill slopes are no steeper than the natural angle of repose. Central-core and sloping core rockfill dams are constructed with embankment slopes ranging between 1V on 2H and 1V on 4H. Rockfill dams with upstream membranes have embankment slopes between 1V on 1.3H to 1V on 1.7H. Concrete-faced and steel-faced rockfill dams usually are provided with the steeper slopes and asphalt-faced rockfill dams have the more gentle slopes to facilitate construction of the asphalt membrane (210).

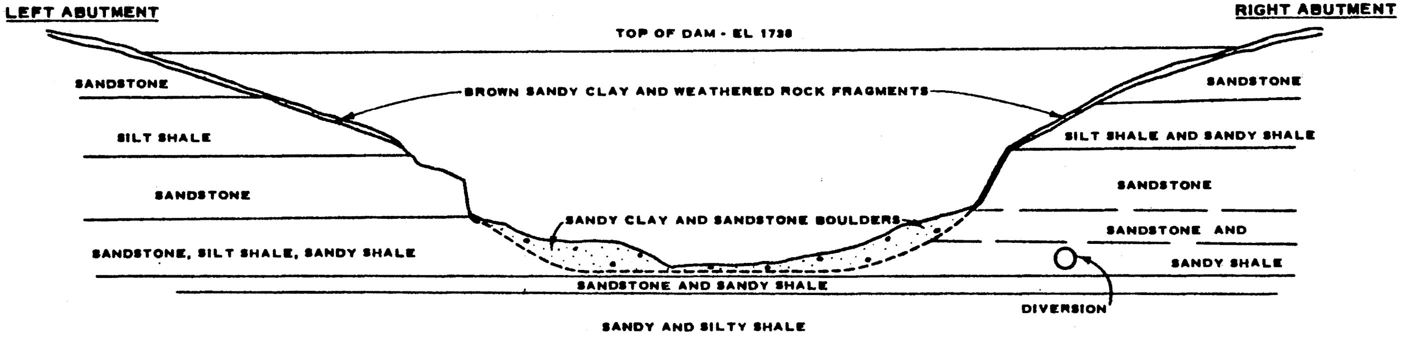
Central Core Dams. The impervious cores of central-core rockfill dams are constructed of fine-grained soils and are flanked by rockfill shells. The major concern during design and construction is to determine and achieve proper gradations within transition zones in order to prevent piping of the impervious core. The downstream transition zones prevent piping of the impervious core during normal reservoir operations and the upstream transition zones prevent piping during reservoir drawdown.

The embankment of Summersville Dam in West Virginia, shown in Figure 24, has a maximum height of 373 feet and consists of a central impervious core flanked by transitions and rockfill shells. The two-zone impervious core was constructed with an upstream zone of silty clay and a downstream zone of silty sand. A zone of weathered sandstone downstream of the core and upstream and downstream spall zones provide transition to the rockfill shells. The rockfill shells were constructed of rolled sandstone in 12-inch and 24-inch lifts, with the smaller lifts and corresponding smaller rock in zones nearer the core. Embankment material was obtained from borrow sources and required excavation (103). The 264-foot high embankment of John W. Flannagan Dam in Virginia, shown in Figure 25, is similar to the embankment at Summersville Dam with a central impervious core flanked by transition and rockfill zones.

<u>Sloping Core Dams.</u> The advantages of a rockfill dam with sloping core are the same as for an earthfill dam with sloping core. An interesting example of a sloping-core rockfill dam, although by no means typical, is provided by Thomaston Dam in Connecticut in Figure 26. In addition to the sloping core of impervious earthfill, Thomaston Dam contains a central earthfill zone of variable sands which provides a transition zone downstream of the core. The upstream and downstream rockfill zones contain weathered granite and mica schist. A unique design feature of Thomaston Dam is the 25-foot wide berm on the downstream slope for a relocated railroad (78).



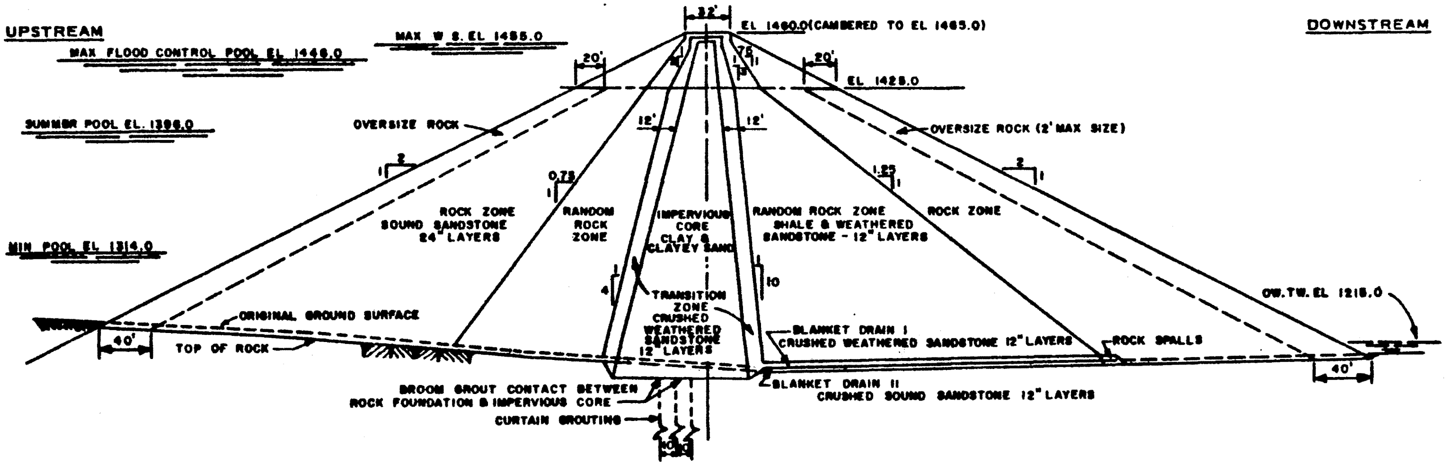
TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (103)

Figure 24. Summersville Dam, Gauley River, West Virginia.



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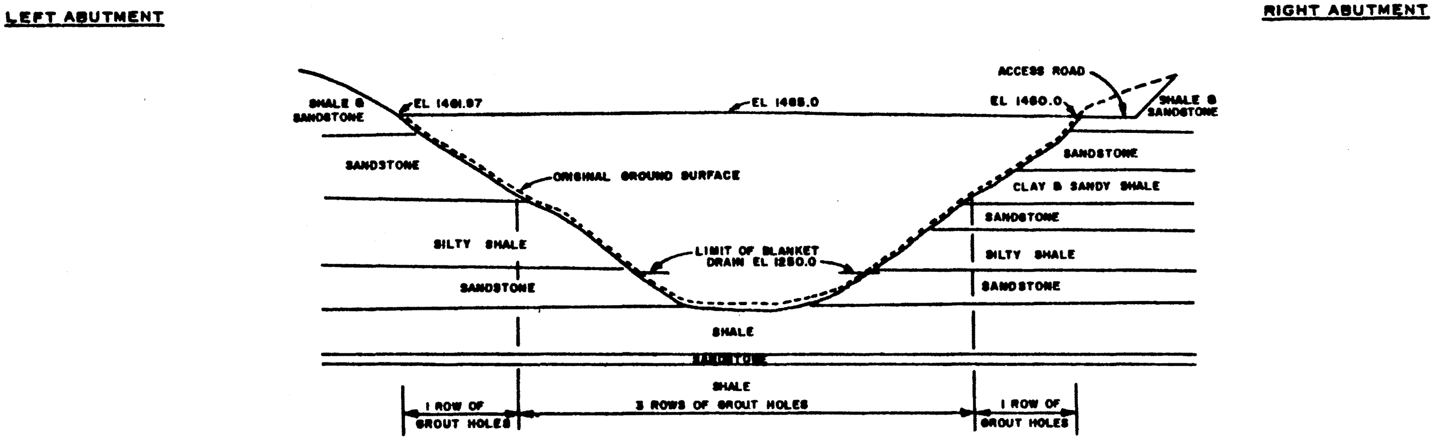
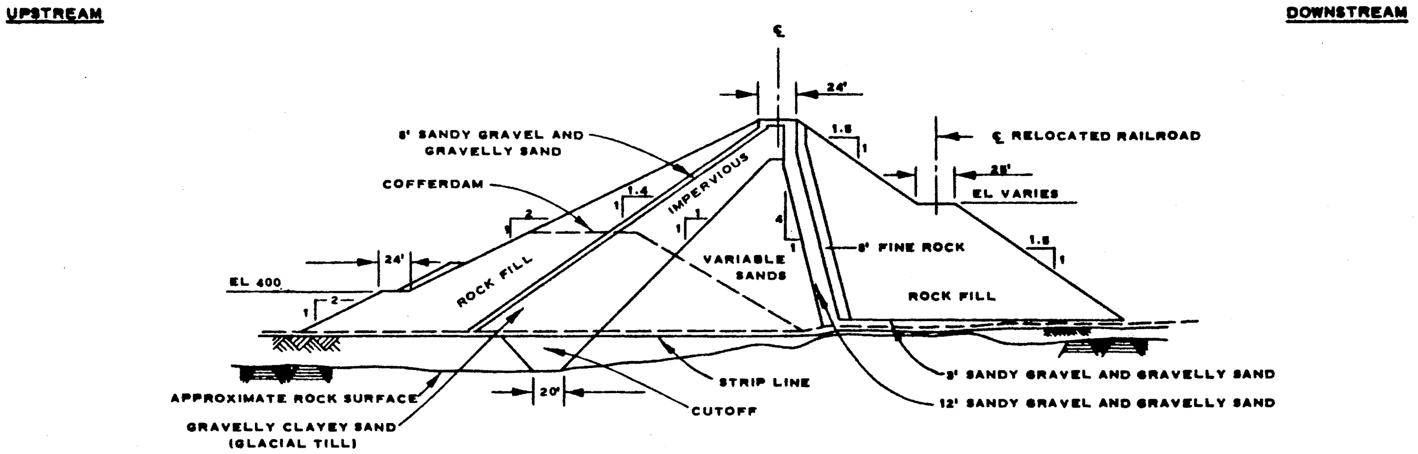


VALLEY CROSS SECTION

Source: (81)

Figure 25. John W. Flannagan Dam, Pound River, Virginia.



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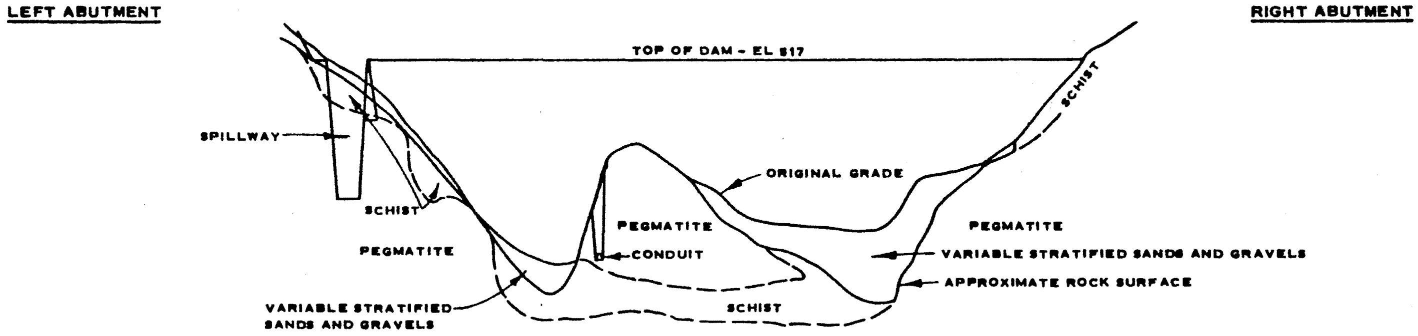


VALLEY CROSS SECTION

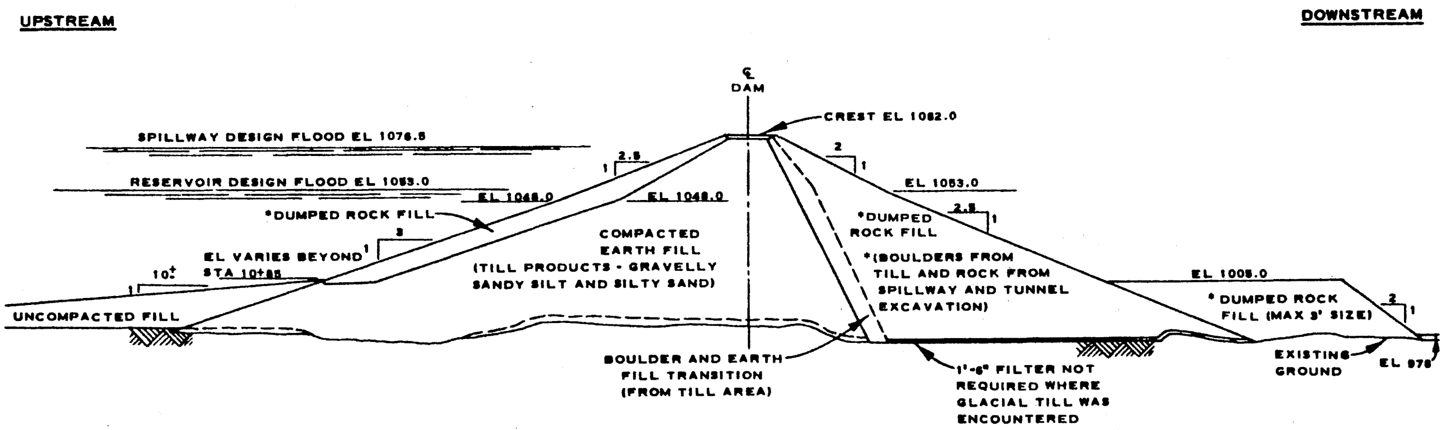
Source: (78)

Figure 26. Thomaston Dam, Naugatuck River, Connecticut.

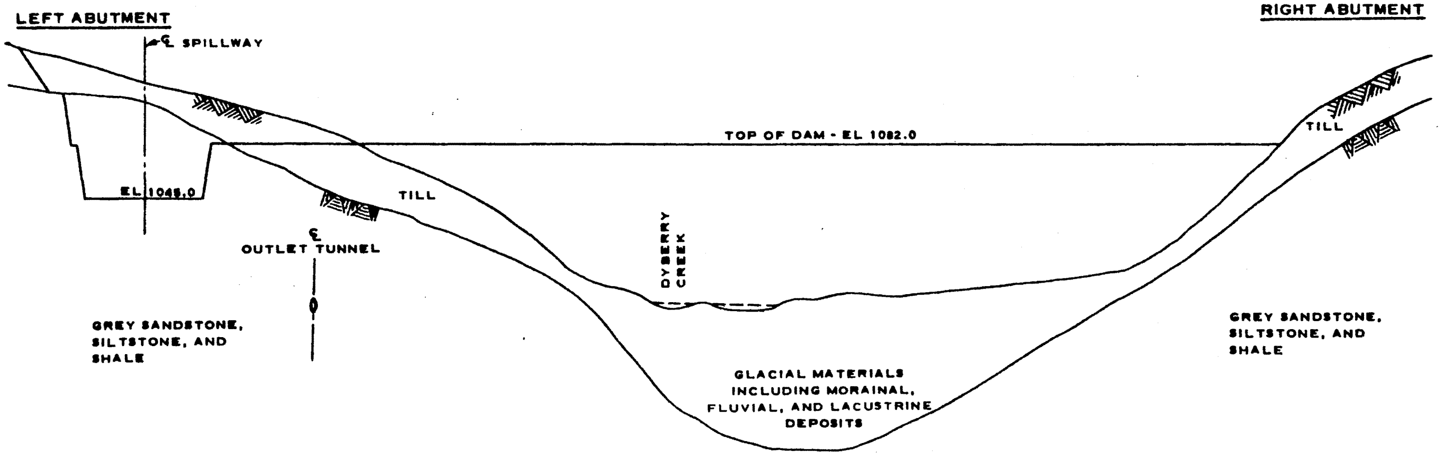
Earth-Rock Dams. A few earth dams fit into neither the earthfill group nor the rockfill group because they are constructed as somewhat of a hybrid of both groups. However, because of the required transition zones to prevent piping of fine-grained impervious material into the downstream rockfill zone these dams are most appropriately referred to as rockfill dams, and perhaps more specifically as earth-rock dams. Two good examples of earth-rock dams are provided by Jadwin Dam in Pennsylvania and Howard A. Hanson Dam in Washington, as shown in Figures 27 and 28, respectively.

Jadwin Dam has a compacted impervious earthfill zone of gravelly sandy silt and silty sand upstream and a dumped rockfill zone of siltstone, sandstone, and shale downstream. The transition zone is 11 feet wide and consists of a mixture of earthfill and boulders. A narrow upstream zone of rockfill provides slope protection (77). Howard A. Hanson Dam has a compacted impervious earthfill zone upstream and a compacted rockfill zone downstream. The earthfill zone was constructed of sand and gravel blended with silty sand topsoil to provide a minimum of three percent by dry weight passing the No. 200 sieve. A gravel drain which varies in thickness from 10 feet at the top to 20 feet at the bottom provides a transition zone between the earthfill and rockfill zones (83).

The failure in 1928 of Schofield Dam, a 62-foot high earth-rock dam in Utah, discouraged for many years designs combining rolled-earthfill zones and dumped-rock zones. Failure of Schofield Dam resulted when large quantities of the upstream earthfill were washed into the voids of the downstream rock zone. Subsequent testing and analysis established conclusively that properly designed filters or



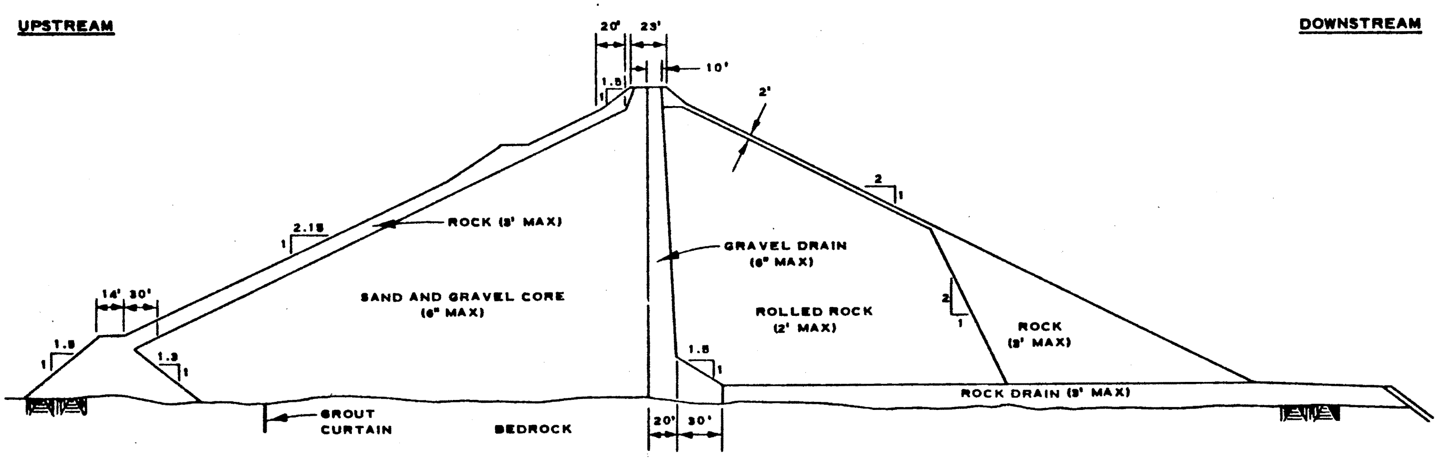
TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (77)

Figure 27. Jadwin Dam, Dyberry Creek, Lackawaxen River, Pennsylvania.



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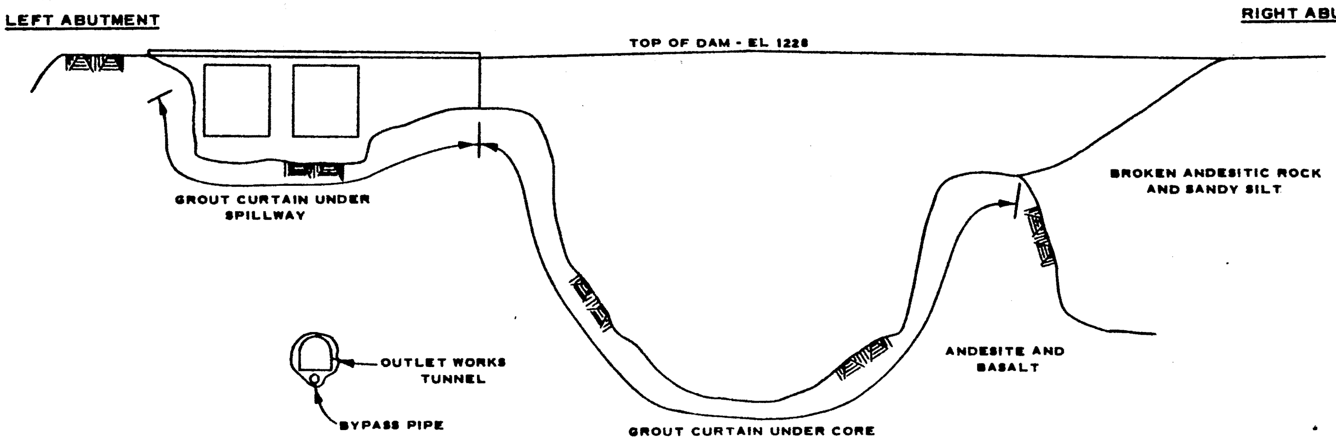


VALLEY CROSS SECTION

Source: (83)

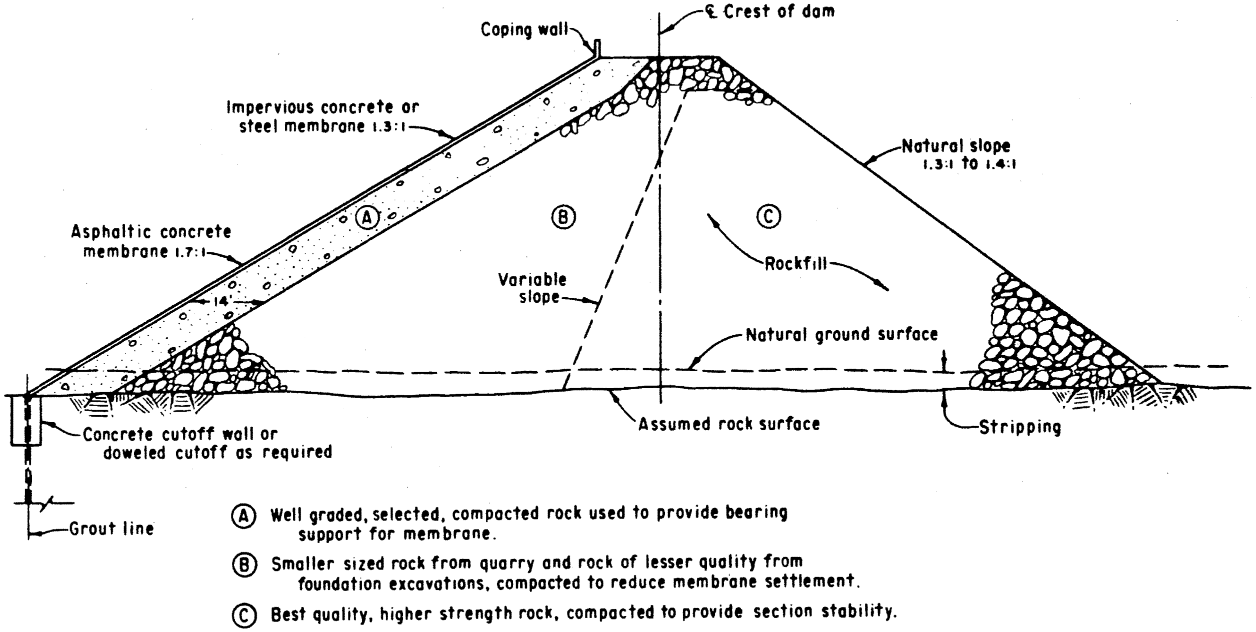
Figure 28. Howard A. Hanson Dam, Green River, Washington.

transition zones provide complete protection against piping failure of the type which occurred at Schofield Dam (57).

Upstream Membrane Dams, Rockfill dams with upstream impervious membranes, or "decked" dams, were the only dams orginally considered rockfill dams. Decked rockfill dams contain rockfill only and grade from coarse rockfill in the downstream zone to finely graded rockfill immediately below the membrane. A typical decked rockfill dam is shown in Figure 29.

Since 1965, concrete-faced rockfill dams have been constructed as compacted rockfill to prevent cracking of the concrete facing, which usually occurred due to consolidation of dumped rockfill. The rockfill zone immediately under the concrete face is crusher-run rock of minus 4 inch to minus 6 inch maximum size, adjusted to provide a gradation with a permeability on the order of 10^{-4} centimeters per second. This rockfill zone is generally about 12 feet in horizontal width at the top and increases moderately in width to the bottom of the dam. The upstream half of all rockfill is placed in 3-foot lifts and the downstream half placed in 6-feet lifts. All lifts are compacted. In current practice, the design face thickness is one foot plus 0.3 percent of the height of the dam. The quantity of reinforcing steel is about 0.4 percent (59).

Rockfill dams with asphaltic concrete membranes are fairly common in Europe and North Africa. However, only two have been constructed in the United States, Montgomery Dam and Upper Blue River Dam, both in Colorado. Steel facing has been used on very few rockfill dams anywhere in the world, and on only one in the United States: El Vado Dam in New Mexico.



Source: (210)

Figure 29. Typical Embankment Section of a Decked Rockfill Dam.

CHAPTER III

EMBANKMENT DESIGN

Design Fundamentals

Seepage

Seepage at Earth Dams. All earth dams are subject to seepage through, under, and around the dam. The progress in time of seepage of reservoir water through the dam depends on the reservoir level and the magnitudes of permeability of the embankment material in the horizontal and vertical directions. (This is essentially Darcy's law for steady-state seepage of water through soil.) If uncontrolled, seepage may adversely affect the stability of the dam due to uplift pressures beneath the dam or subsurface erosion of the embankment by piping.

Determination of Permeabilities. Coefficients of permeability of the compacted earthfill and insitu foundation soils are an estimate of the magnitude of seepage, relative to the hydraulic gradient, which may be expected at a typical section of the dam embankment. Laboratory testing of field samples is the most common method of determining coefficients of permeability (202). Undisturbed samples of insitu soils are essential to determine foundation permeabilities, while borrow materials for embankments should be tested as remolded samples at a compactive effect comparable to that expected during construction.

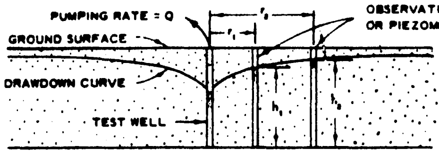
The constant-head permeameter is applicable for pervious soils and the falling-head permeameter is more suitable for fine-grained soils. The methodology of each type is shown in Figure 30.

Field permeabilities as determined by pumping tests differ somewhat from laboratory permeabilities because the natural stratification and structure is not duplicated in the laboratory. Even for relatively homogeneous deposits, field permeabilities are usually 1½ to 2 times greater than laboratory permeabilities (54). The methodology of field pumping tests to determine coefficients of permeability is shown in Figure 30.

Estimation of permeabilities is appropriate for providing preliminary data and rough approximation. Determination from grain-size distribution is applicable only for clean sands and gravels and is illustrated in Figure 30. Figure 30 also presents an approximation of permeability based on soil type.

Analytical Methods. By far the most widely used method to analyze seepage flow through dam embankments and foundations is with the aid of flow nets. Flow nets provide a special solution of the two-dimensional hydrodynamic equation for steady-state seepage (La Place's equation). The solution is obtained by a graphical approach in which all of the spaces formed by intersecting equipotential lines and flow lines are approximately equidimensional (14). The basic requirements and computations of flow nets are shown in Figure 31.

The electric analog method consists essentially of producing and analyzing an analogous conformation in which the actual flow of water through an earth embankment is replaced with an analogous flow of



GRAVITY FLOW (SHOWN ABOVE)

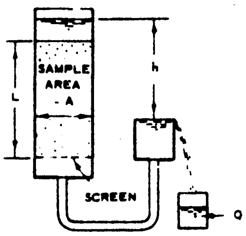
$$k = \frac{Q}{\pi(h_2^2 - h_1^2)} \ln \frac{r_2}{r_1}$$

ARTESIAN FLOW

$$k \; \frac{Q}{2\pi D(h_2 - h_1)} \ln \frac{r_2}{r_1}$$

WHERE D IS THICKNESS OF ARTESIAN AQUIFER

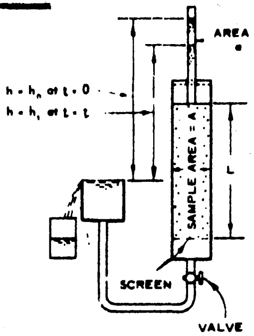
FIELD PUMPING TESTS



$$k = \frac{QL}{hAt}$$

WHERE t = TIME

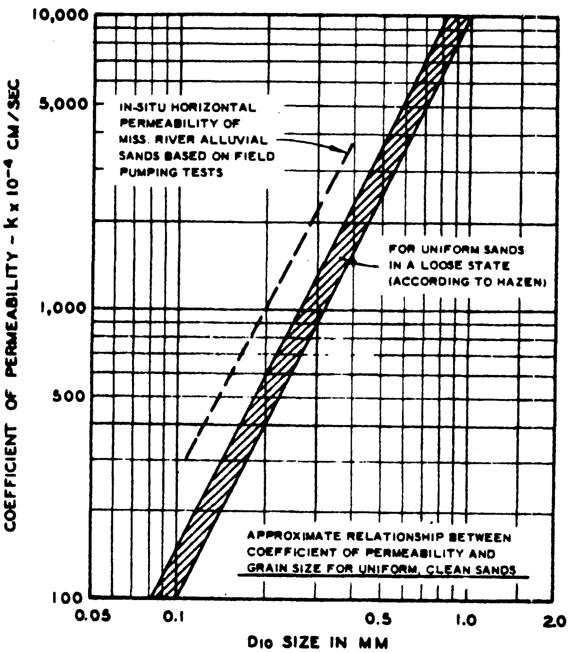
CONSTANT - HEAD PERMEAMETER



$$k = \frac{aL}{At} \ln \frac{h_0}{h_1}$$

WHERE t = TIME

FALLING - HEAD PERMEAMETER



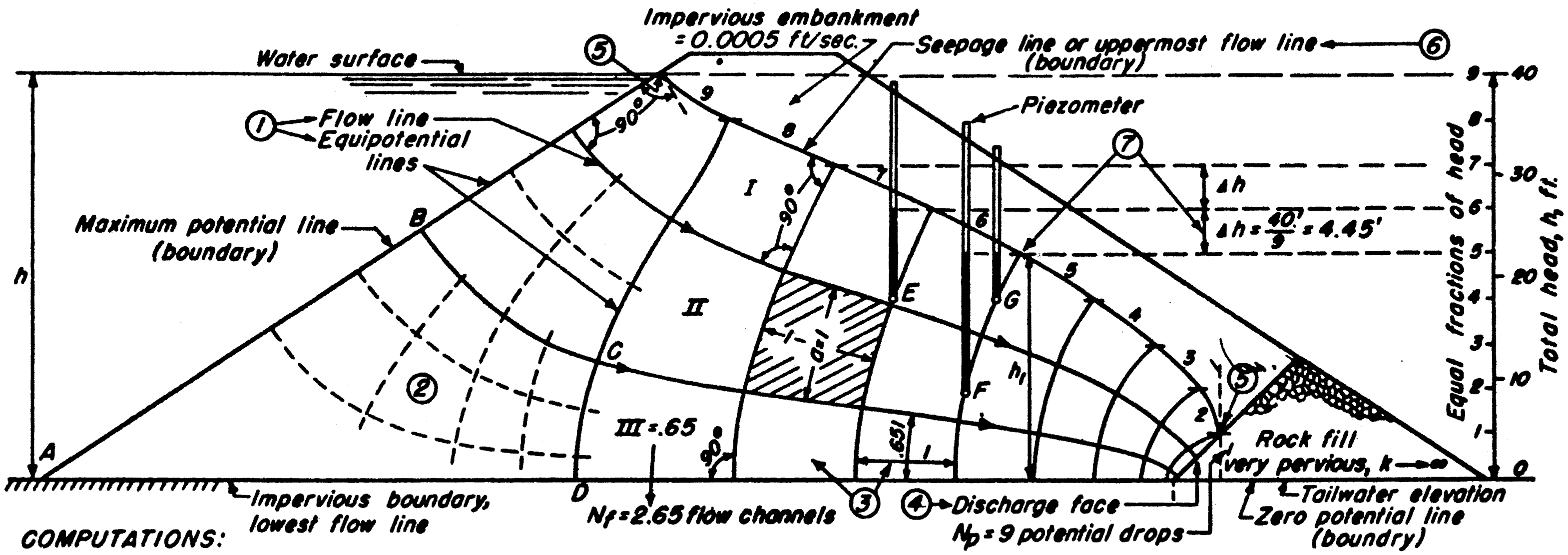
COMPUTATION OF PERMEABILITY FROM GRAIN SIZE

PERMEABILITY AND DRAINAGE CHARACTERISTICS OF SOILS

Coefficient of Permeability, k			Relative Permeability	Soil Type	Method of Determination				
cm/sec	ft/min	ft/yr							
10 1	20 2	10.5×10^6 1.05×10^6	High	Clean gravels Coarse sands	Field pumping tests, reliable if properly executed			Constant-head permeameter, reliable	Computation from grain size
1000×10^{-4} 100×10^{-4} 10×10^{-4}	0.2 0.02 0.002	105,000 10,500 1,050	Medium	Medium sands Fine sands and sand and gravel mixtures Very fine sand		Falling-head permeameter	Reliable		
1×10^{-4} 0.1×10^{-4} 0.01×10^{-4}	2×10^{-4} 0.2×10^{-4} 0.02×10^{-4}	105 10.5 1.05	Low	Silty sands, organic silts Silts, glacial till Silty clay			Unreliable		
100×10^{-9} 10×10^{-9} 1×10^{-9}	200×10^{-9} 20×10^{-9} 1×10^{-9}	105×10^{-3} 10.5×10^{-3} 1.05×10^{-3}	Practically impervious	"Impervious" soils, e.g., homogeneous clays below zone of weathering			Fairly reliable	Computation from consolidation test data (reliable)	

Source: (54)

Figure 30. Drainage Characteristics of Soils and Methods to Determine Permeability.



COMPUTATIONS:

1. Discharge per unit width of section:

$Q = kh\frac{N_f}{N_p} = .0005 \times 40' \times \frac{2.65}{9} = .0059$ c.f.m. per ft. of embankment.

2. Hydrostatic pressure at any point:

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

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

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

h_1 can also be determined graphically

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

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

Source: (205)

Figure 31. Basic Requirements and Computations for Flow Nets.

electricity through a special model. Electric analog methods have not been used as frequently as the simpler and more easily constructed flow nets for analyzing seepage flow at earth dams. However, electric analog methods are more adaptable for the solution of complex seepage problems. Table I gives the analogous relationships between Darcy's Law for seepage flow and Ohm's Law for flow of electrical current, both governed by La Place's equation.

TABLE I

ANALOGY: SEEPAGE AND ELECTRIC FLOWS

Darcy's Law	Ohm's Law
$Q = \frac{KAH}{L}$	$I = \frac{K'A'V}{L'}$
Q = rate of flow of water	I = current (rate of flow)
K = coefficient of permeability	K'= conductivity coefficient
A = cross-sectional area	A'= cross-sectional area
H = head producing flow	V = voltage producing flow
L = length of percolation path	L'= length of current path

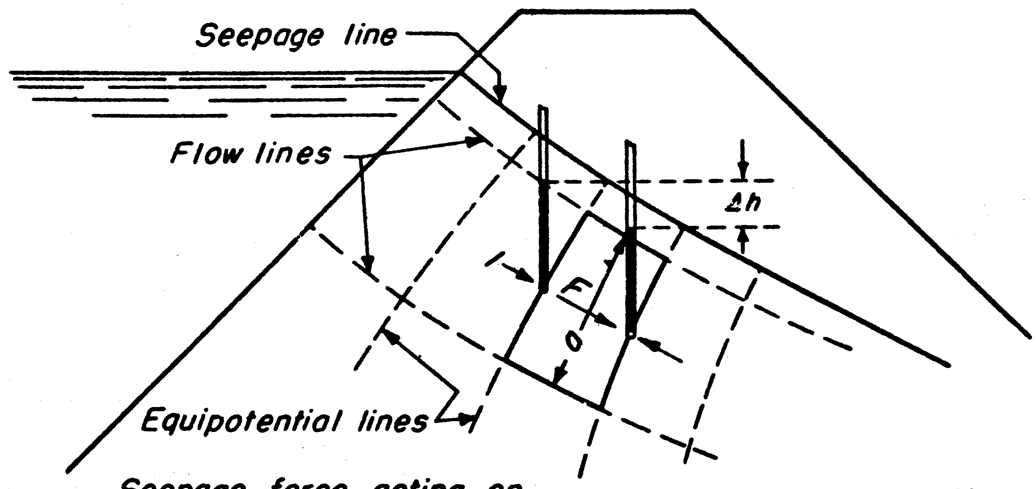
Source: (224)

<u>Seepage Pressures and Piping</u>. When a soil mass is subjected to an upward seepage force of such magnitude that the force exceeds the weight of the soil, failures may occur as heave of the soil mass. Critical conditions, then, may be expected to occur at and beyond the

embankment toe where uplift pressures may exceed the weight of the impervious upper stratum. However, in reality, the upper stratum is invariably interspersed with numerous discontinuities which permit seepage flow along these lesser routes of resistance. Practically all seepage-related failures on record have occurred by subsurface erosion involving the progressive removal of materials through piping; this situation invalidates the concept of failure due to heave (62).

A factor of safety against failure by piping can be calculated through analysis of the resultant seepage force at the expected exit location. The factor of safety is a function of the buoyant weight of a unit element of the upper stratum and the seepage force acting on that unit element. The factor of safety may also be calculated using total unit weight and boundary water pressure. The methodology for determining seepage forces and factors of safety against piping is shown in Figure 32. The extent and location of discontinuities and hence the lines of lesser resistance within insitu soils, and compacted earthfill, cannot readily be determined by practical geological explorations. The factor of safety against failure by piping can then be compared to the factor of safety against failure of a wooden beam which has been weakened to an unknown extent by termites. The usefulness of the factor of safety of the wooden beam cannot be determined by rational procedures (62).

Design of Drains Internal drainage systems provide means to safely collect, convey and discharge seepage from earth dam embankments and foundations, as illustrated in Figure 33. The gradation of the drain material must be sufficiently pervious to allow free passage of



Seepage force acting on segment of unit thickness:

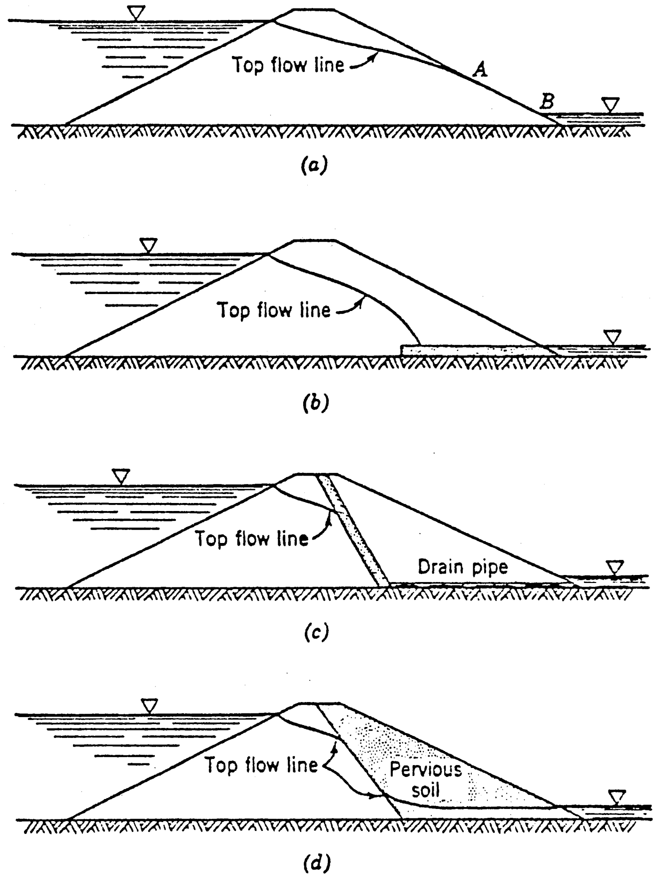
$$F = i A \gamma_w = \Delta h \, a \, \gamma_w$$

Notes:
Seepage force is in the direction coinciding with the lines of flow.

Δh = avg. difference in head between entrance and discharge forces.
l = avg. length of segment.
i = avg. hydraulic gradient = $\frac{\Delta h}{l}$
a = avg. width of segment.
γ_w = unit weight of water.

Source: (205)

Figure 32. Computation of Seepage Force.



Source: (46)

Figure 33. Types of Internal Drainage for Earth Dams: (a) Homogeneous without Internal Drain; (b) Homogeneous with Underdrain; (c) Homogeneous Dam with Chimney Drain; and (d) Zoned Embankment Dam.

seepage through the drain but fine enough to prevent migration of fine-grained earthfill material into the drain. Established filter criteria, Table II, provides guidance on selection of the proper gradation of drain material.

Zones of transition material, or filters, must be provided between embankment zones of greatly differing gradations in order to prevent migration of material from one zone to another. The only recorded failures of earth dams which contained no transition zones between the impervious core and a pervious downstream zone were earth-rock dams which contained a downstream rockfill zone into which impervious material migrated (57). Many earth dams are constructed with a multiple-stage drain system with a distinct, very pervious drainage layer to provide ample pore spaces for water conveyance and a distinct, less pervious filter to prevent migration of fines. The two-stage drainage system, of a filter and a drain, for Warm Springs Dam can be seen in Figure 34. The horizontal filter blanket, which extended up the abutment, was a multiple-stage system consisting of a drain flanked by filters as seen in Figure 35.

The overall integrity of the interface between cohesionless soil and drain material is maintained by the filtering effect of the drain. Minor restructuring of the interface occurs due to viscous drag causing cohesionless soil particles to bridge over pores in the drain (43). The integrity of the interface between cohesive soil and drain material is generally maintained by the cohesive tensile strength of the clay (213). Consequently, the established filter criteria are somewhat too conservative for clays. For protecting medium to highly plastic clays without sand or silt partings, the Corps of Engineers specifies the D_{15}

TABLE II

FILTER CRITERIA

	Bureau of Reclamation	Corps of Engineers
$\frac{D_{15} \text{ of filter}}{D_{15} \text{ of base material}}$	5 to 40	5 or more
$\frac{D_{15} \text{ of filter}}{D_{85} \text{ of base material}}$	5 or less	5 or less
$\frac{D_{85} \text{of filter}}{\text{Max. opening of drain}}$	2 or more	1 or more (round) 1.2 or more (rect.)

The grain size curve of the filter should be roughly parallel to that of the base material.

D_{15} = the size at which 15 percent of the total soil particles are smaller.

D_{85} = the size at which 85 percent of the total soil particles are smaller.

Source: (205)(210)

Source: (123)

Figure 34. Vertical Filter and Drain, Warm Springs Dam, California.

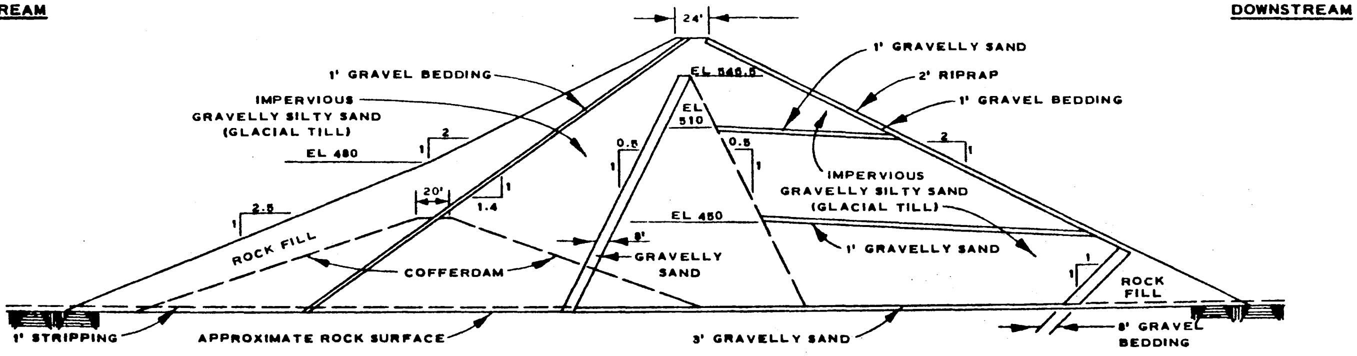
Source: (123)

Figure 35. Abutment Drain Between Filter Zones, Warm Springs Dam, California.

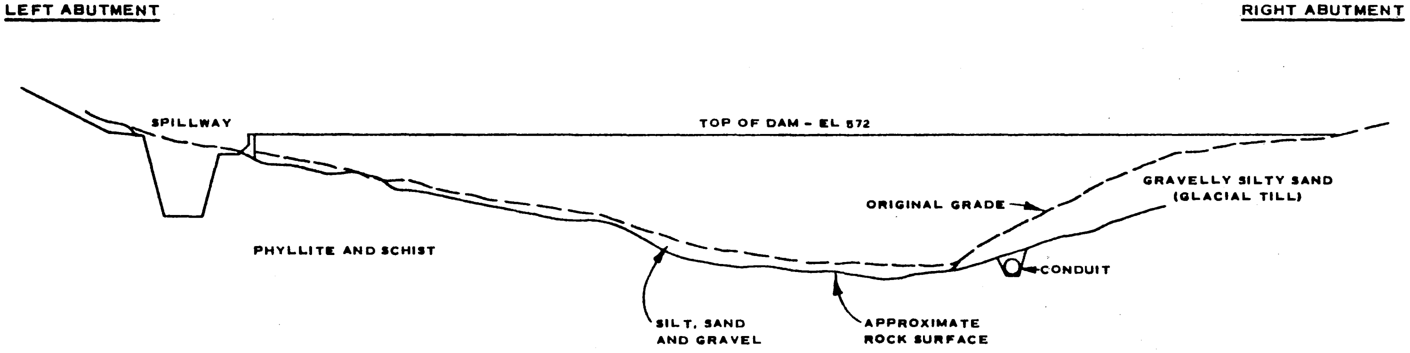
size of the filter to be as great as 0.4 millimeter. The Corps further specifies that the filter must be well graded, with a coefficient of uniformity (D_{60} to D_{10}) not greater than 20 (67). These filter criteria for clay cores may only be adequate for clay cores which remain intact, or free from cracking. The failure of Balderhead Dam in England, in 1967, was due to cracking and the ineffectiveness of the filter to prevent subsequent internal erosion (212). All gradations of samples obtained from the filter of Balderhead Dam were within the gradation limits of the specifications but all were coarser than the upper limit as advocated by the Corps. Subsequently, conservative designs have required silt particles within the filter gradations (213).

Special horizontal drains were constructed within the embankment of North Hartland Dam in Vermont to allow faster dissipation and better control of construction pore pressures (87). The locations of these special drains, as well as the positioning of the inclined drain are shown on the typical embankment section of North Hartland Dam in Figure 36. These special drains were constructed as a result of extreme lateral bulging of the embankment of nearby Otter Brook Dam in New Hampshire which was constructed of very similar earthfill (72). A single upper horizontal filter was constructed in the downstream portion of the embankment of Huntington Dam in Indiana, Figure 37. However, this horizontal filter was not constructed to facilitate pore pressure dissipation, but to provide a filter or transition from the upper random earthfill zone to the lower random rockfill zone which was predominantly blocky limestone (167).

In general, materials selected for filters and drains must be



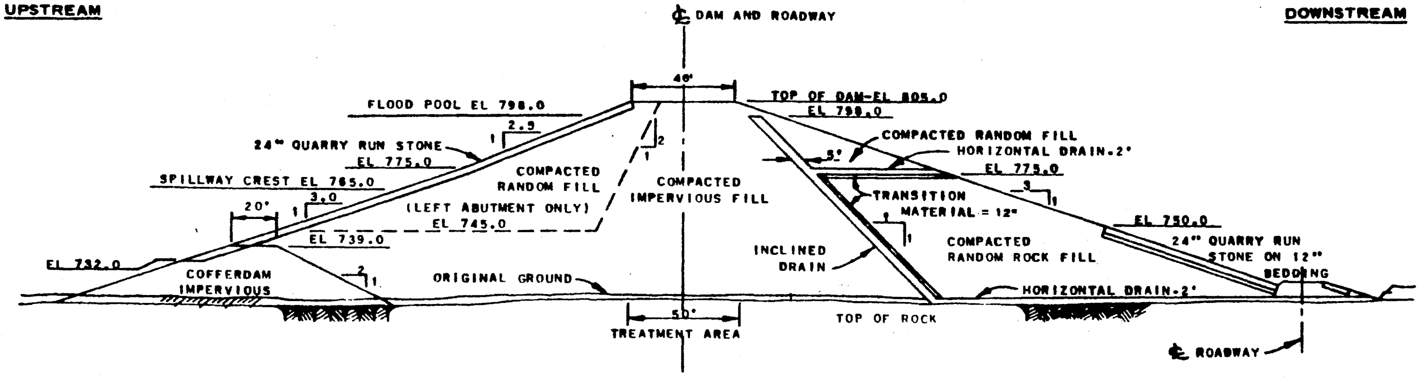
TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (87)

Figure 36. North Hartland Dam, Ottauquechee River, Vermont.



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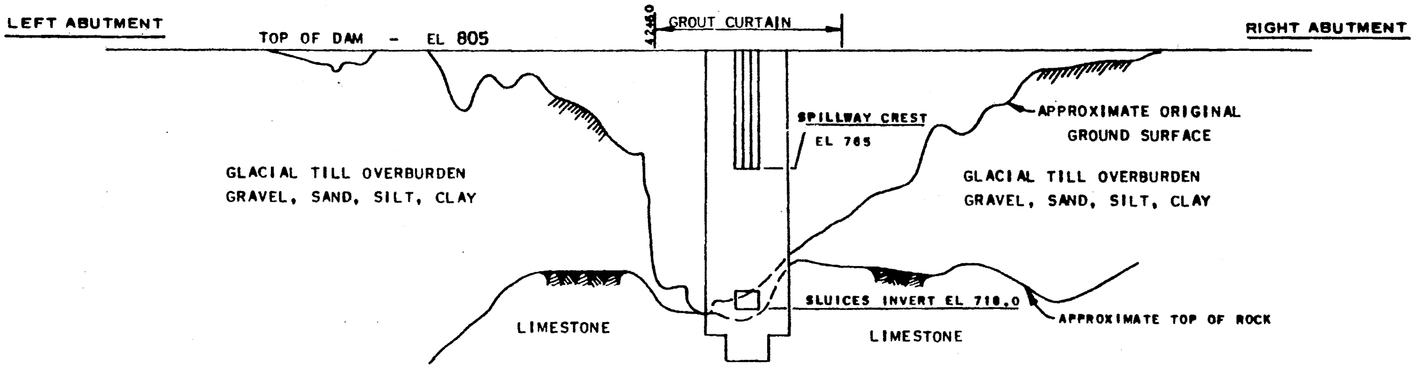


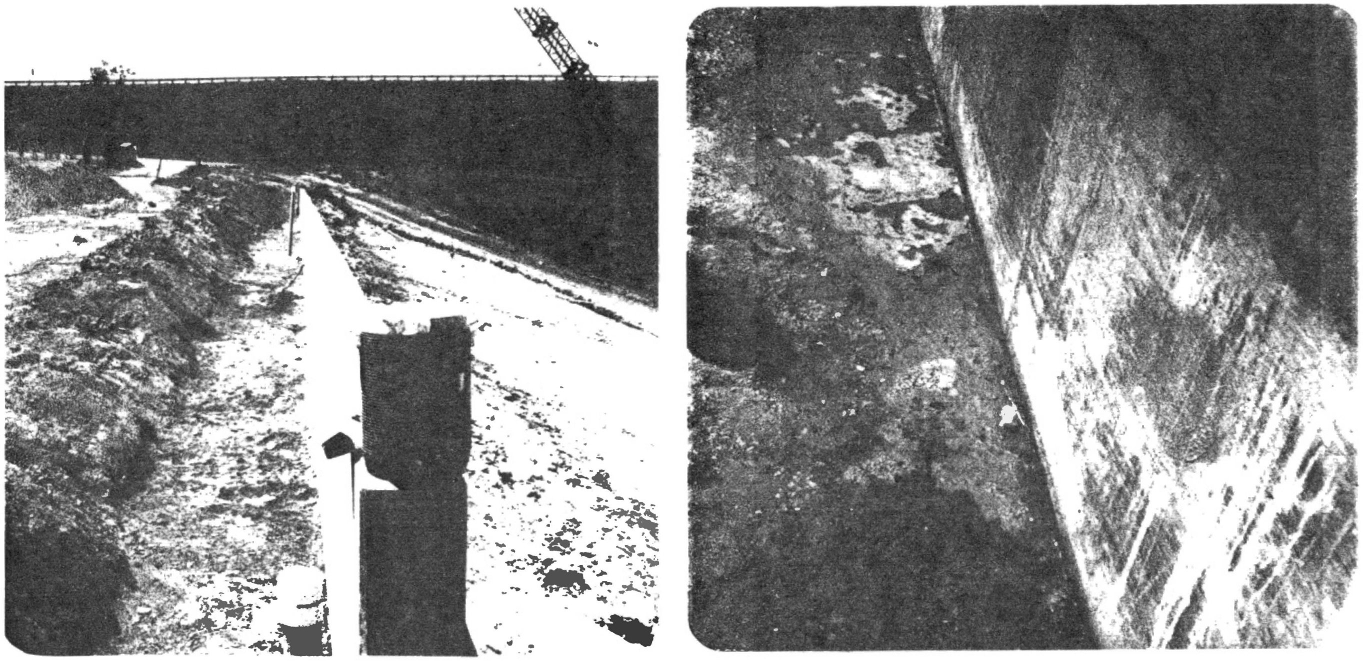
VALLEY CROSS SECTION

Source: (167)

Figure 37. Huntington Dam, Wabash River, Indiana.

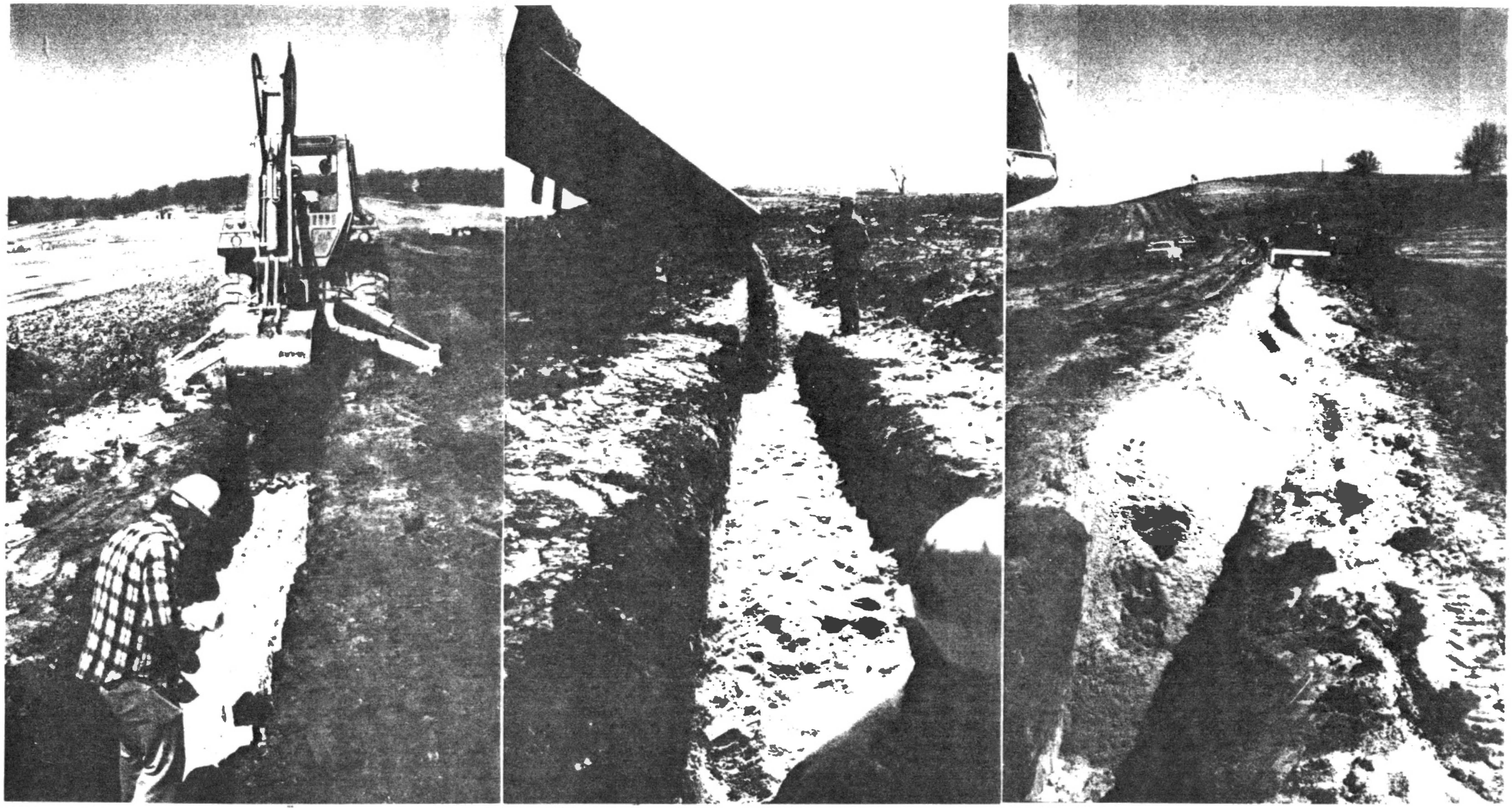
processed to provide satisfactory drainage. Unprocessed materials from natural deposits may result in clogging of the filter in spite of satisfying gradation specifications. A case in point involves recent slope dressing above the powerhouse at Denison Dam. The slope consisted of natural strata of limestone and shale with overlying waste material, again limestone and shale, from the excavation for the powerhouse. Groundwater seepage, complemented with reservoir seepage, had caused sloughing along the slope. Slope dressing required an intercepting filter blanket for control of the seepage discharge prior to placement of random fill and suitable topsoil. A value engineering proposal, submitted by the construction contractor and approved by the Corps of Engineers, allowed substitution of locally excavated material in place of the design filter material since the local material satisfied the gradation specifications. The fines content of the local material did not exceed the maximum allowed by the specified gradation, but did tend to accumulate through migrations with considerable clogging of the filter. Within six months of completion of the slope dressing, the filter failed, causing appreciable sloughing of the new slope (217). The lack of drainage provided by the substitute filter material can be seen in Figure 38.

Inclined filters, drains, and multiple-stage drainage systems are constructed simultaneously with the embankment. The required widths account for access for processing equipment and for contamination fo the edges by construction activities. An optional construction method for vertical single-stage filter-drains involves trenching through embankment material for filter placement, as shown in Figure 39.



Source: (217)

Figure 38. Ineffectiveness of Unprocessed Filter Material Which Met Gradation Specifications, Denison Dam, Texas.



Source: (216)

Figure 39. Construction Steps for Placement of Vertical Filter-Drain, Big Hill Dam, Kansas.

Embankment Stability

Basic Design Considerations. As with any engineered structure, an earth dam must be designed and constructed in such a manner as to safely satisfy structural stability criteria at a reasonable cost. But unlike many engineered structures, an earth dam is subjected to a wide range of loading conditions, the effect of which results to a large extent from the characteristics of the earthfill of the embankment. Additionally, the magnitude of potential damage in the event of a dam failure far exceeds that of most other engineered structures, and therefore the stability of the structure is of extreme importance.

During design, the most accurate stability analyses, and the only appropriate stability analyses for major earth dams, rely on strength parameters developed through triaxial tests of remolded samples from designated borrow areas. Special consideration must be given to possible variation in borrow materials and to inevitable variation in compaction water contents and densities. The decrease in friction angle of granular embankment and foundation materials under high confining pressures must be considered for high dams (3). Other factors which must be considered in order to establish appropriate design parameters include the effect of cracking within the embankment and the compatibility of stress-strain characteristics of differing embankment materials and between the embankment and foundation. Factors influencing design shear strengths are presented in Table III. After completion of the embankment, reanalysis of stability may be accomplished using strength parameters from construction record samples. Design considerations of foundation materials are discussed in later chapters.

TABLE III

FACTORS INFLUENCING DESIGN SHEAR STRENGTHS

FACTOR	INFLUENCE, PERCENT	REMARKS
Conventional rates of shear in laboratory tests	+(5-200)	Effect depends on rate of testing, soil type, rate of consolidation in field, etc.
Progressive failure	+(0-20)	Depends on soil; mainly a facfor foundation soils. May be more serious than shown.
Conventional effective stress design shear strengths		Values shown are for nonfailure or design conditions only; not for failure conditions.
(a) A_f < 1/4 to 1/3 (i.e. embankments)	-(0-30)	
(b) A_f > 1/4 to 1/3 (i.e. foundations)		
Sample disturbance of foundation materials	-(5-20)	Remolding may increase strength of slickensided specimens. Disturbance is greatest for deep borings and soft soils.
Effect of fissures in clays,	+(25-1000)	Generally a factor only for highly overconsolidated soils.
Rough caps and bases in laboratory tests	+5	
Triaxial compression instead of compression, simple shear, extension tests	+(20-30)	Especially important for foundation soils.
Triaxial instead of plane strain tests	-(5-8)	
Back-pressure saturation	Depends on embankment height (significant)	May cause grossly excessive strengths in CU tests at low confining stresses; conservative at high confining stresses.
Conventional plotting of CU test data, as total stress envelopes	-(15-20)	
Isotropic, instead of anisotropic, consolidation in CU tests		Values shown assume test envelopes for isotropic consolidation interpreted as τ_f verses $\bar{\sigma}_{fc}$; in stability analyses.
(a) A_f > 1/4 to 1/3	-(0-30)	
(b) A_f < 1/4 to 1/3	+(0-20)	
Anisotropic material behavior, use of vertical instead of inclined specimens	+(10-40)	

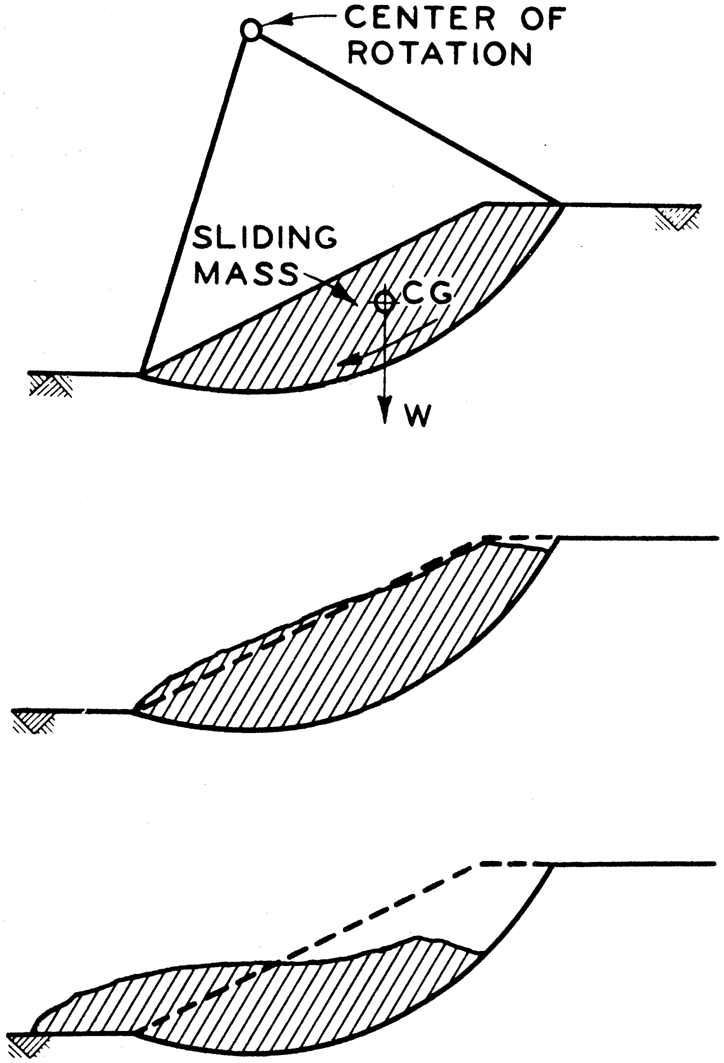
Effect: + = unconservative; causes too high strength
- = conservative; causes too low strength

Source: (34)

Cohesionless Earthfills. The slope stability of dam embankments constructed of cohesionless gravels and sands is dependent upon the friction angle, the slope angle, the unit weight of the earthfill, and seepage pressures. The critical failure mechanism is usually surface ravelling or shallow sliding, which can be analyzed through infinite slope analysis (20). Values of the friction angle can be determined by drained triaxial or direct shear tests, or by correlations with grain size distribution, relative density, and particle shape. Embankment slopes of fine sands, silty sands, and silts are susceptible to erosion by surface runoff. Saturated embankments of cohesionless materials are susceptible to liquifaction and flow slides during earthquakes.

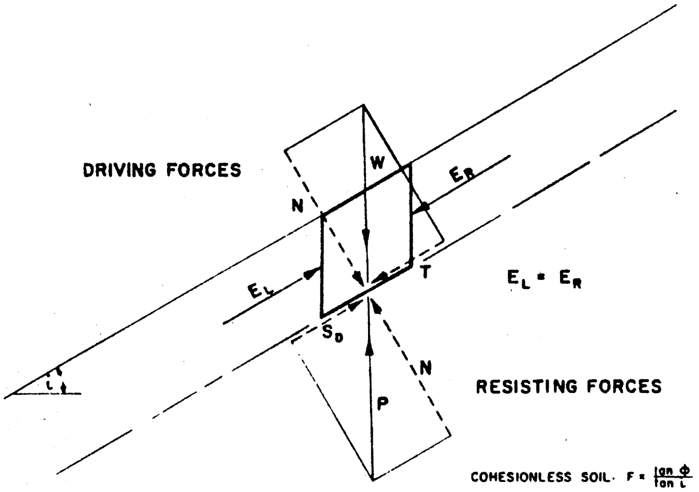
Cohesive Earthfills. The slope stability of dam embankments constructed of clays, clayey sands and gravels, or silts depends on the shear strength of the earthfill as friction angle and cohesion, the unit weight of the fill, the slope angle, pore pressures, and the height of the embankment. The critical failure mechanism is usually sliding on a deep circular surface tangent to the top of the firm foundation, as shown in Figure 40.

Factor of Safety. The factor of safety of an embankment slope is traditionally defined as the ratio of the shear strengths of the material to the shear stress required for equilibrium (21). The rationale for determining a factor of safety is to insure establishing a safety margin to cover uncertainties associated with the measurement of soil properties and with the analysis (31). Although the concept of a factor of safety relates to ultimate failure, the concept should encompass a safety margin against intolerable deformation (45).



Source: (41)

Figure 40. Mechanics of Circular Arc Failure.



Source: (40)

Figure 41. Mechanics of Infinite Slope Analysis.

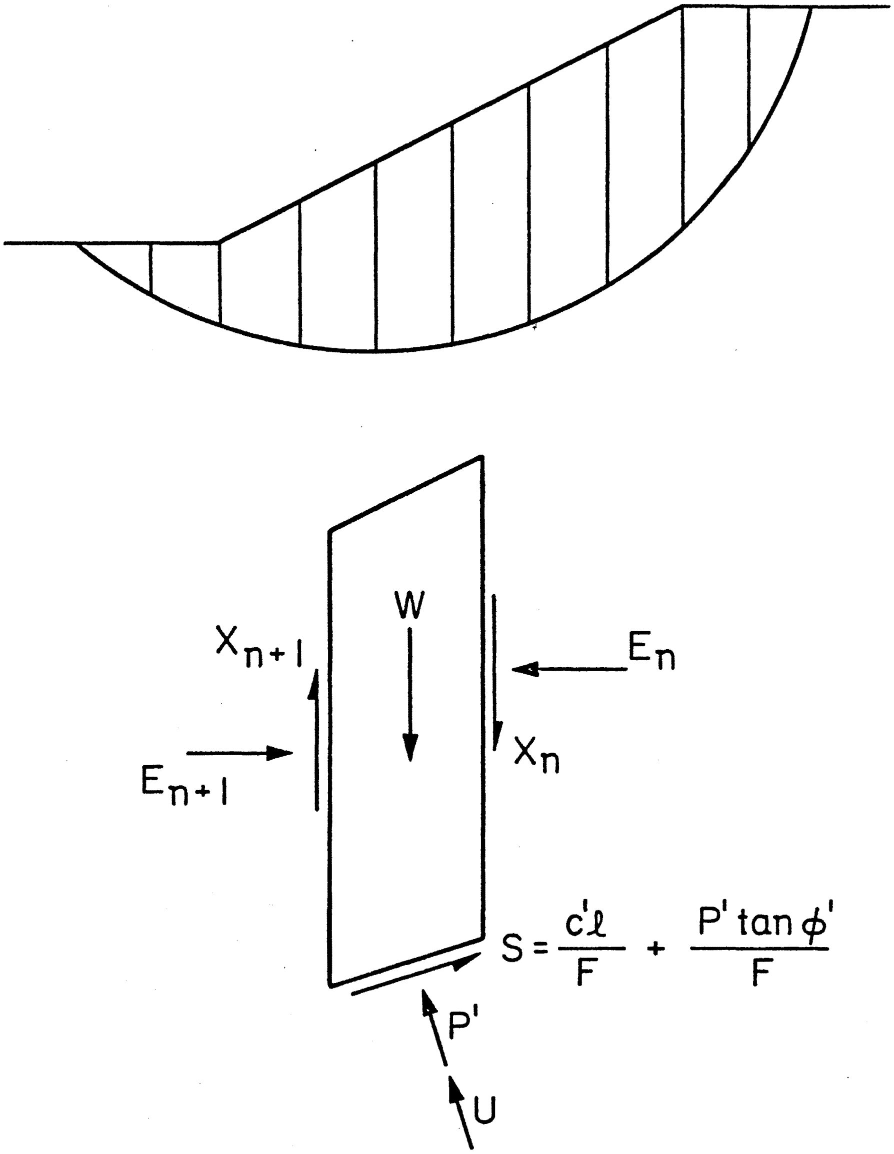
Infinite Slope Analysis. The critical failure mechanism of embankment slopes of cohesionless earthfills can readily be analyzed using the infinite slope method. The term infinite slope designates a constant slope of unlimited extent with constant soil properties, either as a homogeneous soil mass or an anisotropic soil mass with stratifications lying parallel to the slope. Embankment slopes of major earth dams are of sufficient extent to be considered infinite slopes. The forces acting on a unit volume within an infinite slope are presented on Figure 41. Because the infinite slope analysis is very limited to a specific soil type, and since the shear strength of the impervious core influences overall stability of an earth dam, analytical methods involving either a circular arc or a sliding wedge are more adaptable to verifying the stability and establishing a minimum factor of safety.

Circular Arc Analysis. The most common method of stability analysis for earth embankments involves an analysis of a finite number of slices within a circular arc mode of failure. The division of the circular arc into finite slices and the forces acting on one of those slices are shown in Figure 42. A number of limit equilibrium methods have been developed to analyze the overall stability of embankment slopes and the majority of methods is listed in Table IV. These methods of slope stability analysis have four characteristics in common (21): (1) all use the same definition of the factor of safety,

$$F = s/\tau \tag{3.1}$$

in which s is the shear strength of the embankment materials and τ is the shear stress required for equilibrium; (2) all involve the implicit assumption that the stress-strain characteristics of the embankment materials are non-brittle, and that the same value of shear strength may be mobilized over a wide range of strains along the slip surface; (3) all satisfy some or all equations of equilibrium to calculate the shear stress; and (4) all employ explicit assumptions to supplement equations of equilibirum. Methods are classified according to the number of equilibrium equations satisfied, with rigorous methods satisfying all equations of equilibrium. The rigorous methods are much too complex and time-consuming to be used for design purposes and are used primarily in research. An evaluation of the more commonly used methods of stability analysis is presented in Table V.

The finite element method of analysis incorporates the nonlinear stress-strain characteristics of embankment material into stability analyses. And unlike limit equilibrium methods which assume a constant shear stress and a constant mobilized shear strength along the slip



Source: (19)

Figure 42. Forces on an Individual Slice for Stability Analysis Methods Employing Slices.

TABLE IV

EQUILIBRIUM CONDITIONS SATISFIED BY VARIOUS PROCEDURES OF STABILITY ANALYSIS

PROCEDURE OF ANALYSIS	EQUILIBRIUM CONDITIONS SATISFIED						SLIP SURFACE
	Overall			Individual Slices			
	Moment	Vertical Force	Horiz. Force	Moment	Vertical Force	Horiz. Force	
(ϕ = 0) Method	Yes	(Yes)	(Yes)	Not a Slices Procedure			Circular Arc
Logarithmic Spiral	Yes	(Yes)	(Yes)	Not a Slices Procedure			Log Spiral
Plane Shear Surface, Culmann	Yes	Yes	Yes	Not a Slices Procedure			Plane
Friction Circle Method	Yes	Yes	Yes	Not a Slices Procedure			Circular Arc
Frohlich	Yes	Yes	Yes	Not a Slices Procedure			Circular Arc
Bell	Yes	Yes	Yes	Not a Slices Procedure			General Shape
Ordinary Method of Slices - Fellenius	Yes	No	No	No	No	No	Circular Arc
Petterson	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape
Fellenius Rigorous Graphical	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape
Raedschelders	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape
Modified Bishop	Yes	(Yes)	No	No	Yes	No	Circular Arc
Bishop Rigorous	Yes	(Yes)	(Yes)	Yes	Yes	Yes	Circular Arc
Nonveiller	Yes	(Yes)	(Yes)	(Yes)	Yes	Yes	General Shape
Spencer	Yes	(Yes)	(Yes)	Yes	Yes	Yes	General Shape†
Morgenstern and Price	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape
Janbu et. al. - Horiz. Side Forces	No	(Yes)	(Yes)	No	Yes	Yes	General Shape
Lowe and Karafiath	No	(Yes)	(Yes)	No	Yes	Yes	General Shape
Corps of Engineers - Modified Swedish Method	No	(Yes)	(Yes)	No	Yes	Yes	General Shape
Janbu Generalized Procedure of Slices - (CPS)	(Yes)	(Yes)	(Yes)	Yes	Yes	Yes	General Shape
Seed and Sultan	No	(Yes)	(Yes)	No	Yes	Yes	Two Sliding Wedges
Corps of Engineers - Sliding Block	No	(Yes)	(Yes)	No	Yes	Yes	Three Sliding Blocks

Source: (222)

TABLE V

EVALUATION OF STABILITY ANALYSIS PROCEDURES

Characteristic	Remarks
Results achieved	(1) For c large and ϕ small: All methods give about same result, including ordinary method of slices. (2) For c small and ϕ large: Ordinary method of slices is too conservative. Others give about the same result. (3) For circles extending beyond toe: Taylor-Lowe gives slightly high F.
Practicality of hand computation	Approximate order of preference: Ordinary method of slices, modified Bishop method, Taylor-Lowe.
General suitability for most purposes with circular surfaces and computer solution	Approximate order of preference: Use any method except ordinary method of slices; modified Bishop, Taylor-Lowe.
For evaluating proposed methods	Use Morgenstern-Price.

Source: (34)

surface, the finite element method calculates the stress and moblized strength at points along the slip surface. The factor of safety from a finite element analysis is the summation of the increments of available shear strengths relative to the summation of the increments of shear stress, or

$$F_{average} = \frac{\Sigma(c + \sigma \tan\phi)}{\Sigma \tau} \tag{3.2}$$

The finite element method is very complex and not at all adaptable to providing stability analyses for design purposes. However, the finite element method provides an excellent tool to compare the merits of various limit equilibrium methods. Combining four parameters into one dimensionless parameter results in more easily developed comparisons (32). This dimensionless parameter incorporates the embankment height (H), the strength parameters (c and ϕ) and the unit weight of the soil (γ), as

$$\lambda_{c\phi} = \frac{\gamma H \tan\phi}{c} \tag{3.3}$$

The remaining parameters are the slope angle (β) and the magnitudes of pore water pressures within the slope. The pore pressure ratio, r_u, relates the pore pressure to overburden pressure, as

$$r_u = \frac{u}{\gamma H} \tag{3.4}$$

Table VI presents calculated factors of safety from the finite element method along with calculated factors of safety from the more commonly used limit equilibrium methods. These particular comparisons are for an embankment slope of 1 vertical on 2.5 horizontal and for a pore pressure ratio of 0.6.

TABLE VI

COMPARISON OF MINIMUM VALUES OF FACTOR OF SAFETY

Slope: 1V-on-2.5H Pore pressures (r_u = 0.6)	$\lambda_{c\phi}$				
ANALYSIS PROCEDURE	0	2	5	20	50
Finite Element	1.00	1.00	1.00	1.00	1.00
Log Spiral	1.00	0.98	0.99	0.99	1.01
Ordinary Method of Slices	1.00	0.91	0.88	0.82	0.83
Modified Bishop Method	1.00	0.98	0.98	0.97	0.99
Force Equilibrium	1.08	1.02	1.01	1.01	1.01
Spencer's Procedure {f(x) = constant}	1.00	0.98	0.99	0.99	1.01
Morgenstern & Price {f(x) = constant}	1.00	0.98	0.99	0.99	1.01

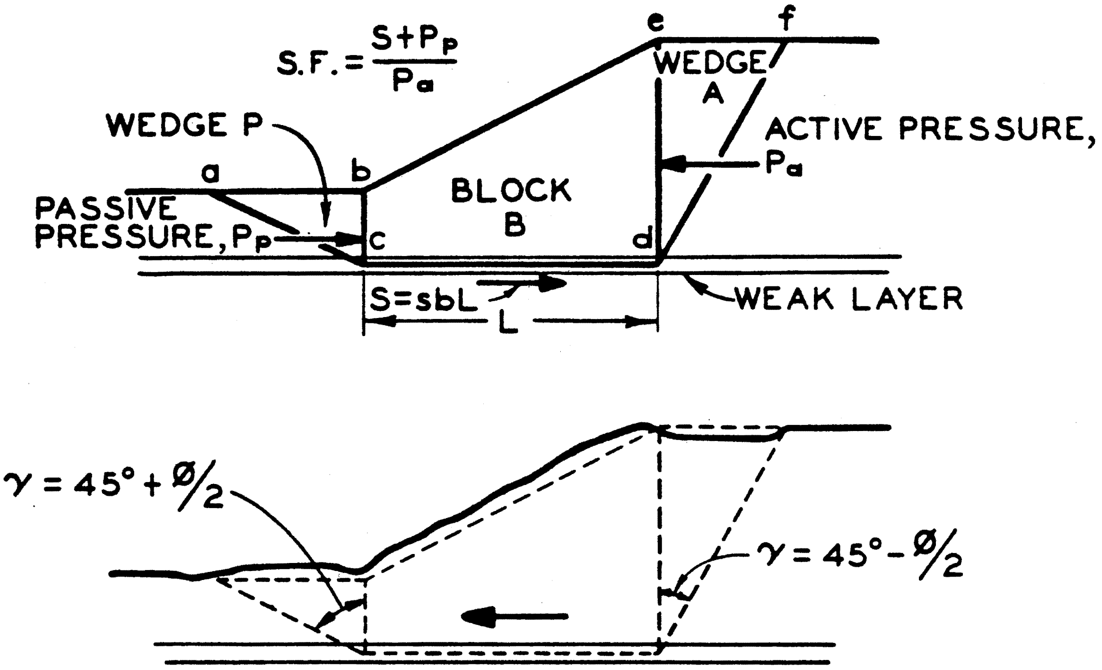
Source: Adapted from (18)(20)

The comparisons in Table VI show the accuracy of various methods of stability analysis, including the complex finite element method, and most methods are within ± 5 percent. Comparisons utilizing different slope angles and pore pressure ratios verify this accuracy over a wide range of applications (19)(21). With the exception of the ordinary method of slices, which is used only for preliminary purposes, the uncertainties related to assumptions and approximations of the various stability methods are far exceeded by the uncertainties related to soil properties (21) (31). Therefore, an analytical stability procedure should be selected for its simplicity and speed of analysis; which explains why the modified Bishop method is the most commonly used procedure.

Wedge Analysis. The sliding wedge method, included in the evaluations and comparisons as force equilibrium methods, provides stability analyses for specific applications. The wedge method is ideally suited for analyses of zoned embankments, particularly with inclined zone interfaces, and for analyses of stratified soil foundations. The characteristics of a slide along a foundation plane analyzed as a critical wedge is shown in Figure 43.

Embankment Cracking

Mechanics of Cracking. Cracking develops within zones of tensile stresses within earth dams subjected primarily to differential settlement. The pattern of cracking may develop transversely with the axis, longitudinally, or diagonally. Most commonly, cracks form in vertical planes, but may vary in inclination to horizontal planes. The extent



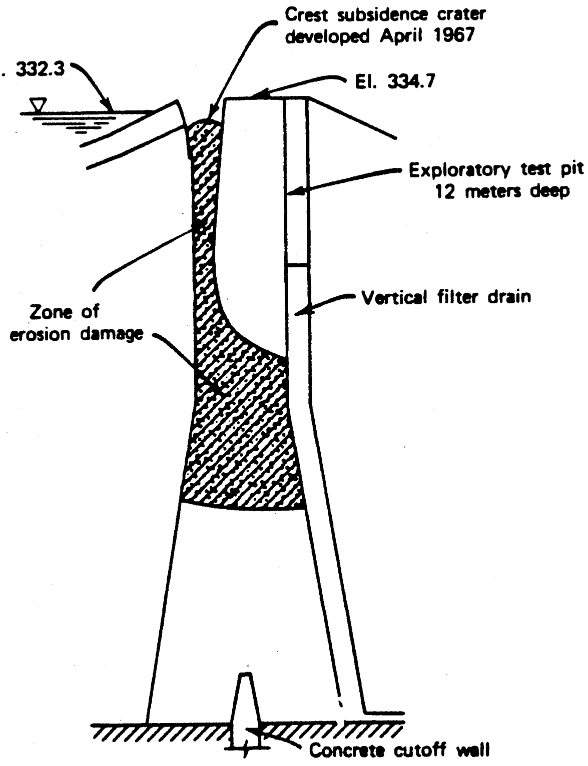
Source: (41)

Figure 43. Mechanics of Wedge Analysis.

of a crack may be localized or may be continuous through the width of an earth dam. Zones of tensile stresses are limited, by the very nature of the phenomenon, to embankment zones of cohesive soils. Tensile stresses cannot develop within cohesionless soils and therein lies their value as filters to prevent piping of impervious core materials through cracks within the dam embankment. Differential settlement, due to differing foundation compressibilities provides the precipitating factor for the most dangerous cracking: transverse cracking of the impervious core.

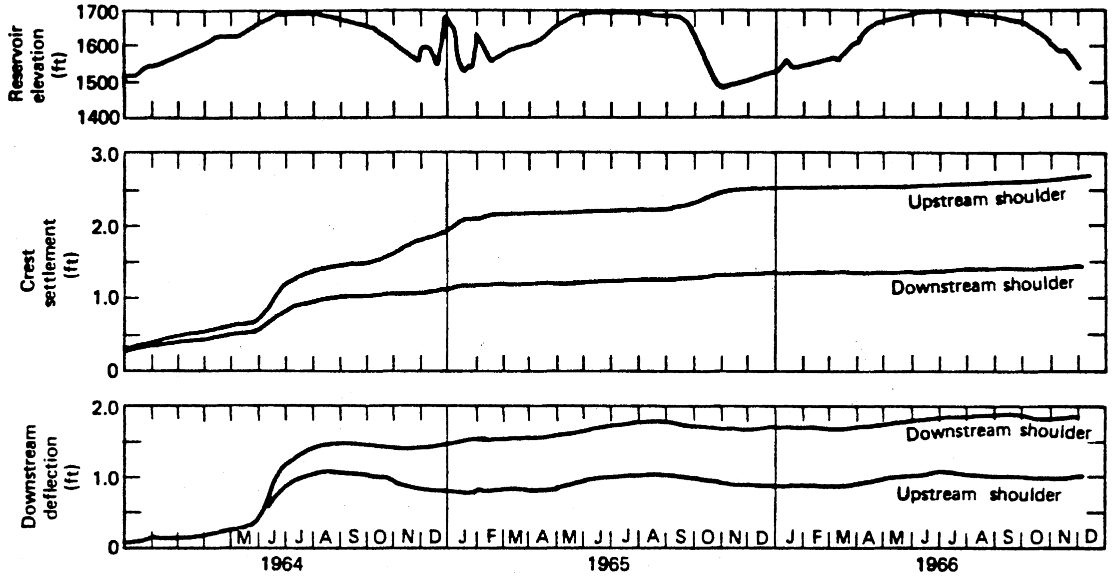
Transverse Cracking. The most dangerous cracking within an earth dam is transverse cracking of its impervious core or impervious zone. The danger of transverse cracking lies in the creation of flow paths for concentrated seepage through the dam embankment. Transverse cracking develops within zones of tensile stresses which occur as a result of differential settlements within foundations of differing compressibilities. At locations where differential settlements occur over relatively short horizontal distances, with small radii of curvature, the tendency toward development of tension zones will be a maximum (128). The more common locations are at steep abutments where a short transition exists between near-surface incompressible rock of the abutments and deep, compressible overburden soils of the valley; at the junction of a closure section and a previously constructed embankment section; and over old stream channels or meanders filled with highly compressible soils. If the channel crosses the embankment alignment diagonally, the pattern will occur as a diagonal crack. Transverse cracking, or more appropriately, the adverse effects of transverse cracking, is negigible in the lower portions of high earth dams because the extreme pressures of overlying embankment fill tend to keep cracks closed in spite of reservoir-induced hydraulic pressures. Therefore cracking presents potential hazards in the upper portions of high dams and in dams of low to intermediate height. Balderhead Dam in England failed due to piping of the impervious core through a transverse crack (212). The zone of erosion damage at Balderhead Dam is shown in Figure 44.

Longitudinal Cracking. Longitudinal cracking also develops within zones of tensile stress within earth embankments. However, in addition to differential settlement within the foundation, differential settlement within the embankment and lateral deformation of the embankment may create longitudinal zones of tensile stresses in which cracking may develop. Compacted earthfill within cutoff trenches through compressible overburden soils provides a zone of less compressible material to cause differential settlement within the foundation. Embankment zones of differing material and compressibilities may be subject to differential settlement within the embankment, as illustrated by Cougar Dam in Figure 45. Lateral deformation of the embankment due to increasing fill height during construction may result in longitudinal cracking, as shown at Skiatook Dam in Figure 46. Longitudinal cracks do not provide open seepage paths, as transverse cracks do, and therefore are of less consequence in regards to piping through the embankment. However, longitudinal cracks reduce the overall embankment stability and may promote slope failure or sloughing, particularly if the cracks fill with water, from storm runoff for example.

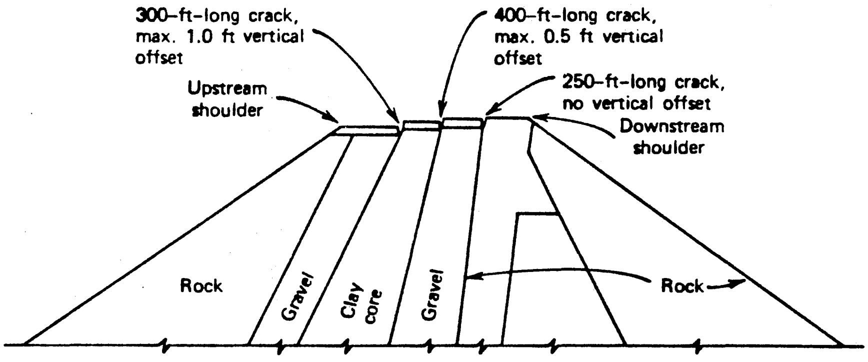


Source: (212)

Figure 44. Extent of Erosion Damage by Piping to Impervious Core of Balderhead Dam, England.



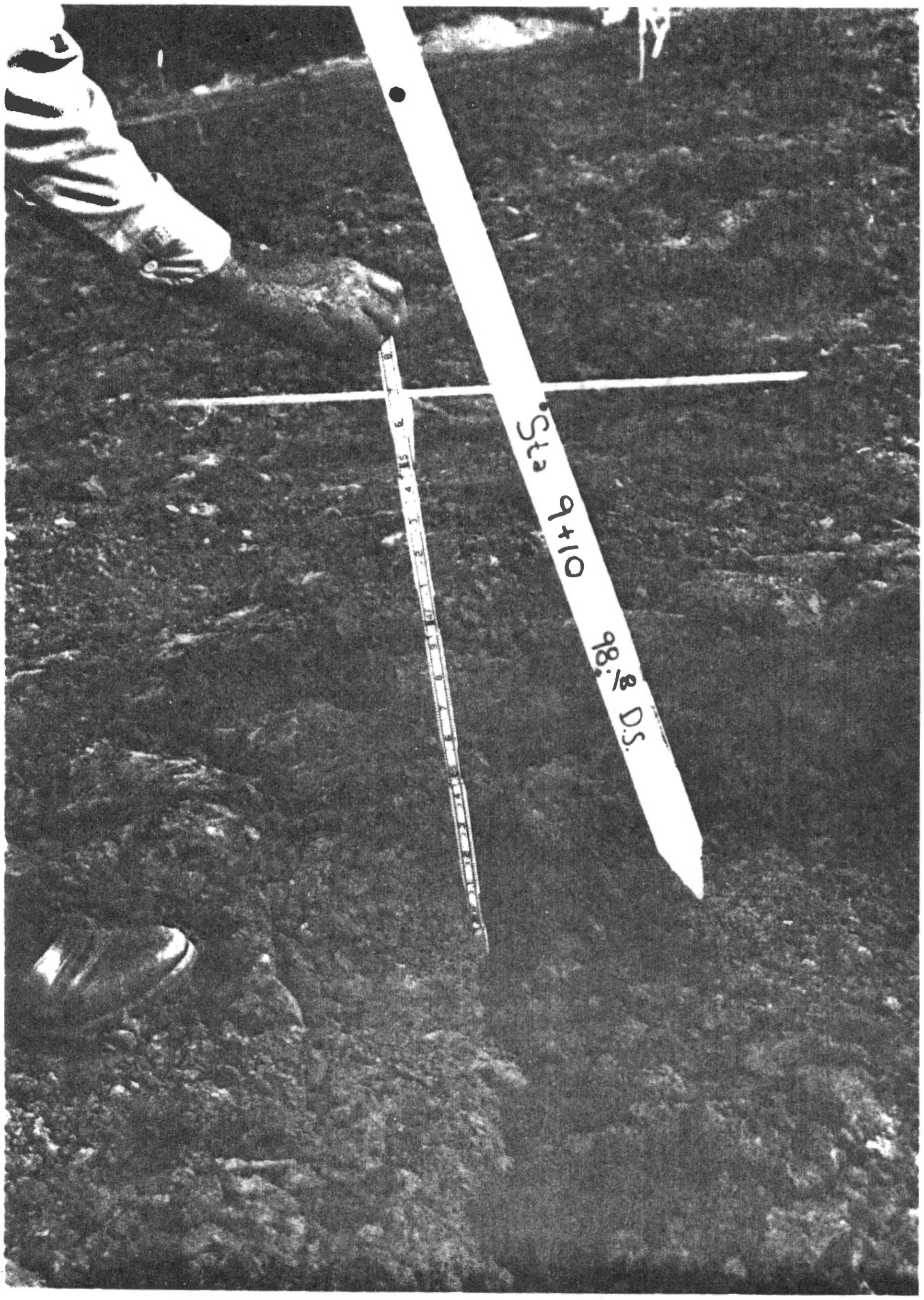
(a) Settlement and Crest Deflection.



(b) Longitudinal Cracking at Crest.

Source: (49)

Figure 45. Post-Construction Settlement and Cracking at Cougar Dam, Oregon.



Source: (218)

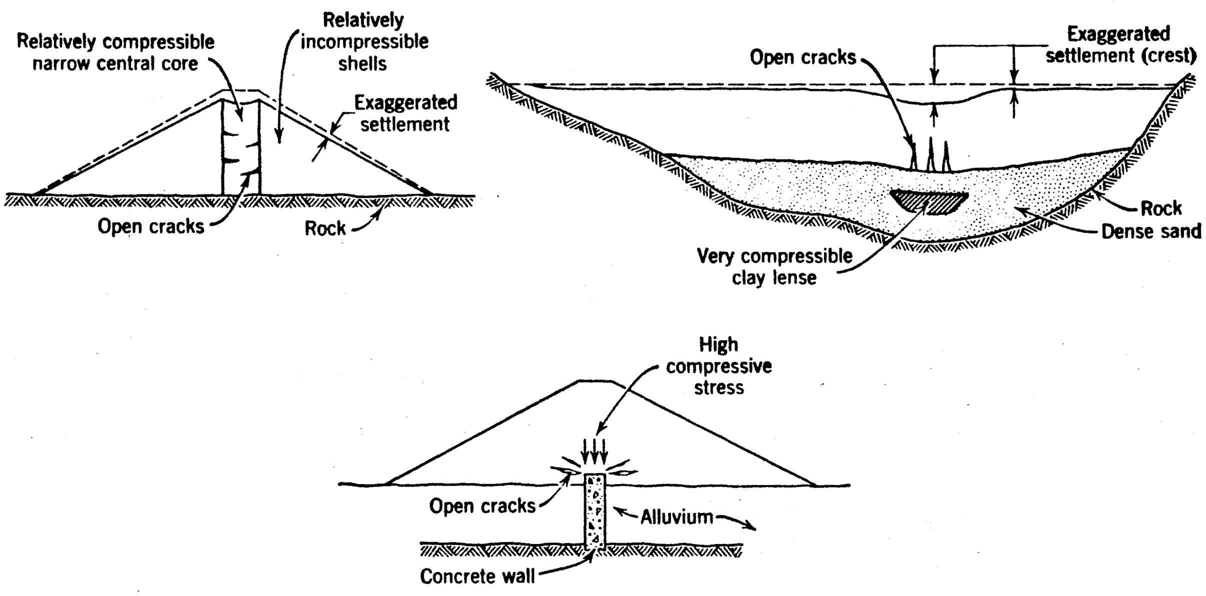
Figure 46. Longitudinal Cracking During Construction of Skiatook Dam, Oklahoma.

Interior Cracking. Cracking within the interior of the embankment that never extends to the surface may only be inferred, unless forensic investigation after failure of a dam uncovers evidence of interior cracking. While interior cracking also develops within zones of tensile stresses resulting from differing compressibilities of adjacent materials, this cracking is usually of lesser extent. Some locations of interior cracking are along the interfaces between embankment zones of differing soil properties, above small isolated lenses of highly compressible foundation materials, and next to major heterogeneities such as concrete walls. These examples are illustrated in Figure 47. Interior cracking adjacent to outlet works conduits may result in piping along the conduit and failure of the dam (57). Interior cracking may occur as a result of incompatibility of the stress-strain characteristics of the embankment and foundation and have an adverse effect on overall stability. This particular type of interior cracking may be expected to occur once the embankment is constructed beyond the critical height. An approximate value of this critical height can be determined from Figure 48. The stability of embankments which are constructed beyond this critical height should be analyzed by assuming that the embankment is cracked to a depth of

$$H_c = \frac{4c}{\gamma} \tan (45 + \phi/2) \qquad (3.5)$$

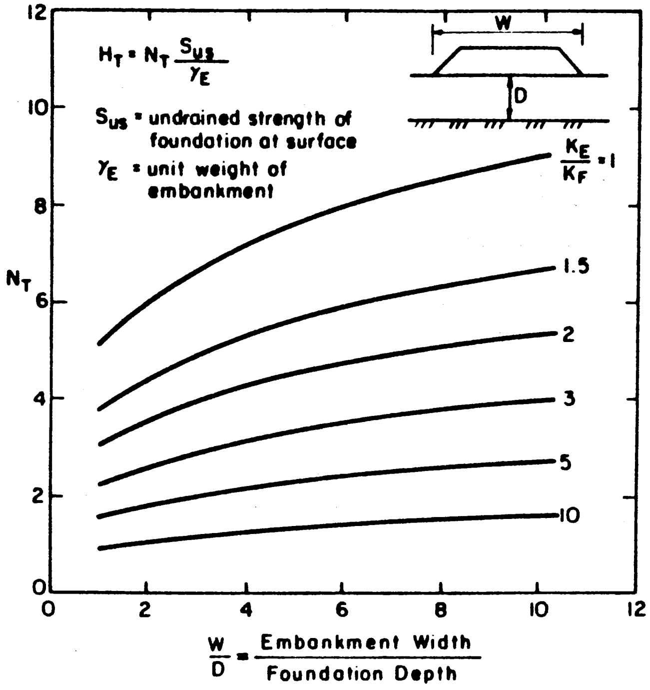
in which H_c is the crack depth, c and ϕ are strength parameters and γ is the unit weight of the embankment material (20).

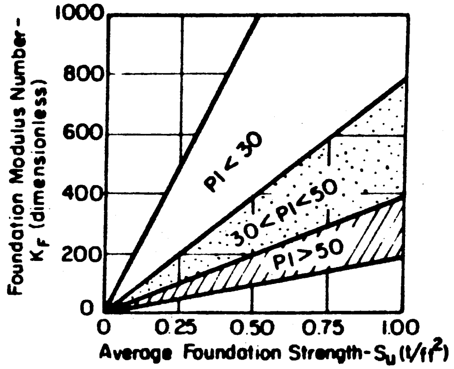
Horizontal Cracking. The precipitating factors of horizontal cracking are somewhat different from the other types of cracking, but



Source: (57)

Figure 47. Examples of Interior Cracking.





Typical values of K_E for compacted fills

Unified Class.	Compaction Water Content		
	Optimum - 3%	Optimum	Optimum + 3%
GC	300 - 1200	200-500	75 - 300
SP	400 - 1000	400-1000	400-1000
SM	300 - 750	300 - 750	300 - 750
SC	250 - 1000	150 - 600	50 - 250
ML	250 - 1000	150 - 600	50 - 250
CL	250 - 1000	100 - 400	30 - 200
CH	100 - 400	50-200	20 - 100

Values shown apply to fill materials compacted to dry densities from 90% to 95% of the Std. AASHO maximum. In general, the value of K_E increases with increasing dry density at a given water content.

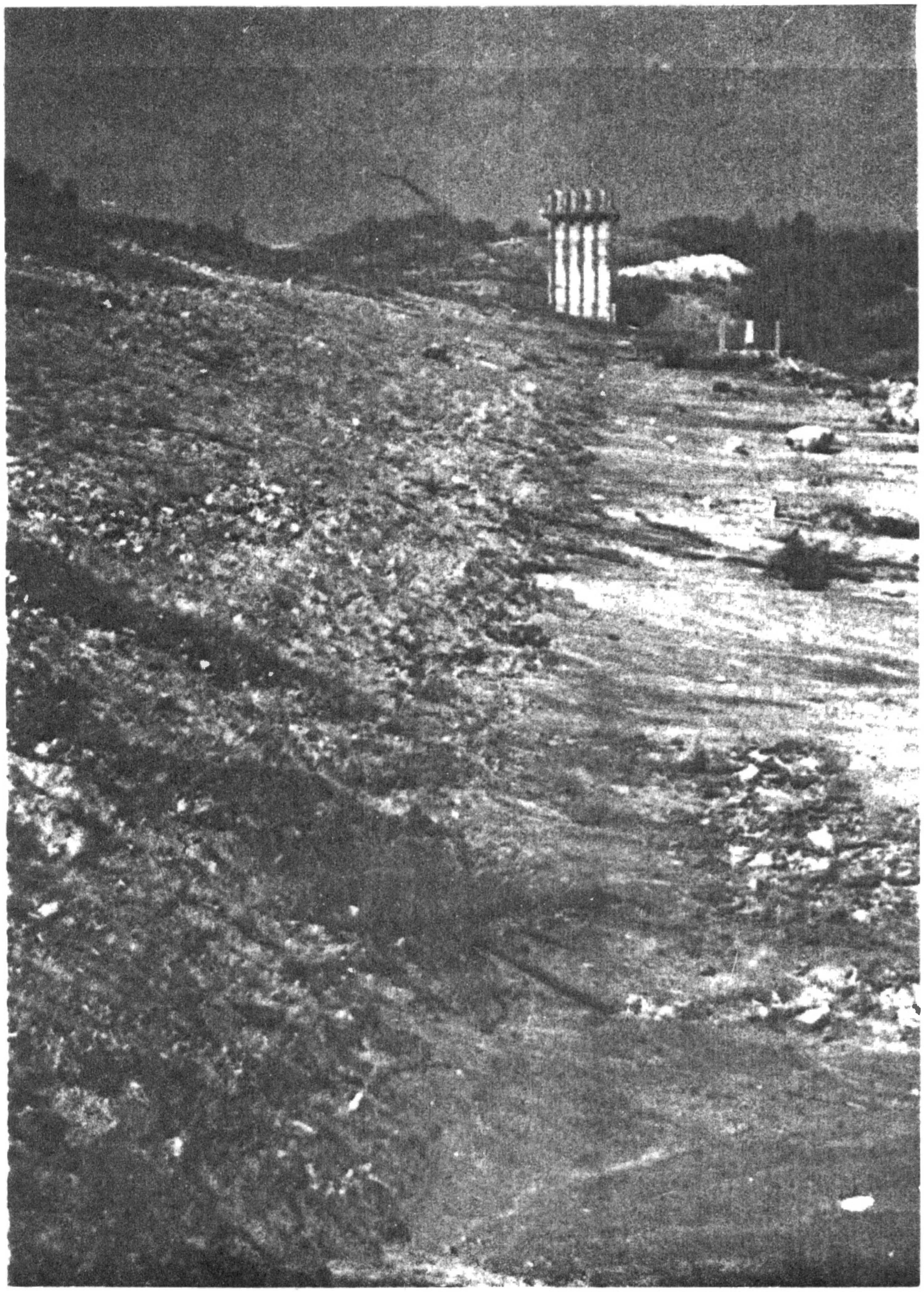
Source: (20)

Figure 48. Method of Estimating Critical Embankment Height for Cracking.

are included because of the similarities in appearance. Horizontal cracking implies a readily visible surface crack along a horizontal plane in the embankment, and is not to be confused with horizontal interior cracking. Horizontal cracking is a result of overthrust of the embankment relative to the foundation, or of an upper portion of the embankment relative to a lower portion. The overthrust occurs during construction due to incompatibility of stress-strain characteristics between the embankment and the foundation or between an upper portion of the embankment and a lower portion that was compacted at a different moisture content, or allowed to dry appreciably during a lengthy construction halt (219). The embankment overthrust at Birch Dam and the accompanying horizontal crack are shown in Figure 49. Shallow cracking may occur within wind-dried surface "skins" due to lateral bulging of embankments. Horizontal cracking, in itself, is of minor importance. Overthrust cracking reseals itself through remolding of materials along the thrust plane (140).

Defensive Design Measures. Embankment design and construction should provide measures which eliminate or significantly reduce differential settlement and subsequent embankment cracking. However, in spite of design and construction measures to prevent cracking, defensive design measures should be implemented to mitigate potential damage from possible cracking. A partial listing of design and construction measures are included:

1. Irregularities along abutments should be eliminated; high spots removed and low spots filled in with dental concrete. The inclination of the abutments is of less significance than irregular-



Source: (5)

Figure 49. Embankment Overthrust at Birch Dam, Oklahoma.

ities as evidenced by the satisfactory performance of earth embankments adjacent to concrete sections in dams (55).

2. The upper portion of the impervious core can be constructed as a more plastic material by progressively increasing the compaction water content and progressively decreasing the compactive effort, as at Warm Springs Dam (123). Materials susceptible to cracking are presented in Figure 50.

3. Stage construction may be employed to provide an initial surcharge for partial consolidation of compressible foundations. The limits of initial stages could coincide with the limits of compressible foundation material. High dams would provide extensive pressure within cracked initial stages to prevent seepage through cracks (128).

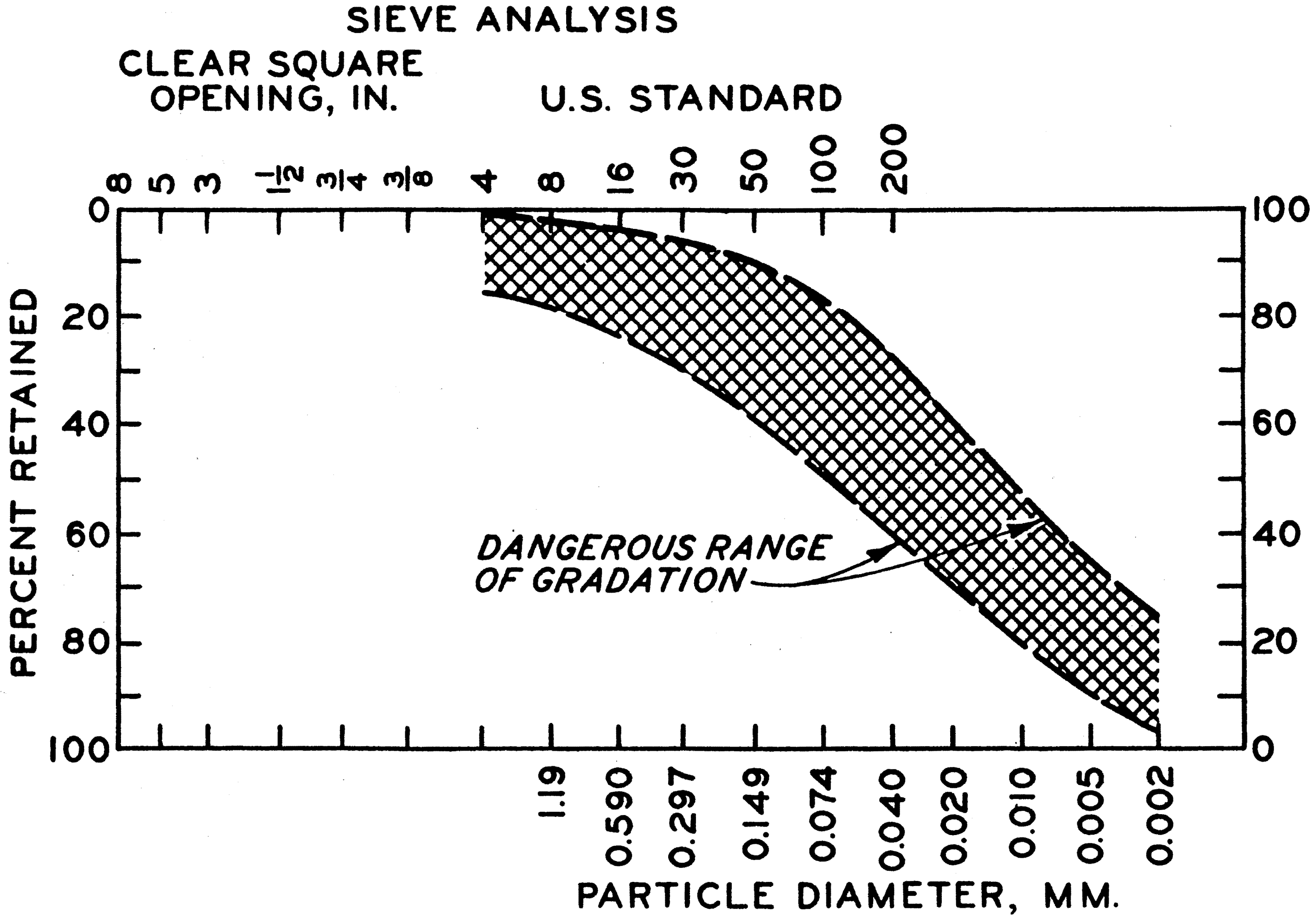
4. The end slope of the embankment at the closure section should be 1 vertical on 5 horizontal to reduce differential settlement of the closure section embankment (128).

5. Thin cores should be avoided within dams prone to cracking. Wider cores obviously require more extensive cracking to provide a continuous seepage path (55).

6. Longitudinal cracks should be incorporated into stability analysis of embankments constructed of cohesive fill overlying compressible foundations (20).

7. Wide filters or transition zones should be provided downstream of impervious cores, as well as amply large drains. The gradation of the filters is an important concern; Balderhead Dam failed due to piping along a transverse crack because the filter gradation was too pervious to retain the impervious material of the core (213).

8. Well-graded material upstream of the impervious core to be



Source: (57)

Figure 50. Gradation of Materials Which are Susceptible to Cracking.

carried into cracks as a "crack stopper" may be advisable (51).

9. The density of pervious shells and impervious cores should be as great as possible to reduce settlement through saturation by the reservoir and through consolidation (128).

10. Arching the dam upstream may cause compressive stresses in the embankments after the reservoir is impounded, however, there is considerable disagreement on this point (57).

When Cougar Dam in Oregon was completed in 1964, it was the highest earth dam (rockfill) ever constructed. Because of the unprecedented height and steep abutments, the possibility of transverse cracks developing within the impervious core due to reservoir impoundment was recognized during design. Consequently, a wide transition zone was provided downstream of the impervious core as a secondary, semi-impervious zone which would prevent erosion of core material in the event of cracking, and would control the leakage. Also, the axis was arched upstream, and the abutments were shaped beneath the impervious core to reduce tensile stresses which might develop within the core as a result of settlement and downstream deflection. Additional compactive effort was provided in the downstream rockfill zone to reduce settlements and deflections to a minimum (92).

Compaction Characteristics of Embankment Materials

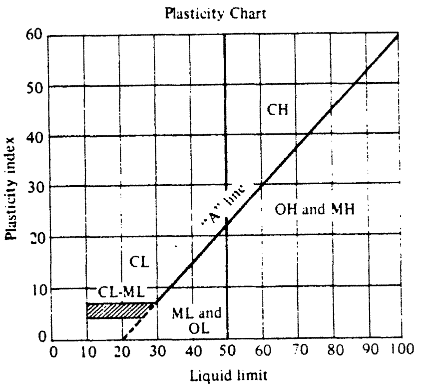
Classification of Materials

The earth materials of which a dam embankment is constructed are unique to a particular site and may range from massive rock to silt and clay. For the purposes of analyzing potential borrow materials for construction, these materials are grouped according to three broad classifications with each classification considered separately. One classification is cohesive soils and consists of those soils designated by the Unified Soil Classification System, Figure 51, as fine-grained soils and coarse-grained soils with appreciable fines. Another classification is cohesionless soils and consists of coarse-grained soils with little or no fines. The last classification is rockfill. The general engineering properties of cohesive and cohesionless soils used in dam embankments are presented in Table VII.

Cohesive soils can be substantially altered through compaction in order to provide more satisfactory embankment properties. Cohesionless soils and rock require compaction to increase density and reduce compressibility. Generalized compaction curves of cohesive soils and sand are shown in Figure 52. The maximum dry density of uniform fine sand occurs at very slight moisture contents; the dry density then decreases with increasing moisture content. For silt, a very sharp-peaked curve of dry density versus moisture content usually occurs. For clays, the curve is less sharp and tends to become flatter with increasing plasticity of the clay.

Compaction Fundamentals

Compaction is a mechanical process by which a soil mass consisting of solid soil particles, air, and water is reduced in volume by temporary applications of loads. Compaction reduces the volume of the soil mass by reducing the volume of air through expulsion; the volume of water is not significantly reduced. Therefore, the water content of a

Major Divisions			*Group Symbols*		*Typical Names*	*Laboratory Classification Criteria*		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 per cent GW, GP, SW, SP More than 12 per cent GM, GC, SM, SC 5 to 12 per cent *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM[a]	d u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits below "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d u	Silty sands, sand-silt mixtures		Atterberg limits above "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less, the suffix u used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

Source: (210)

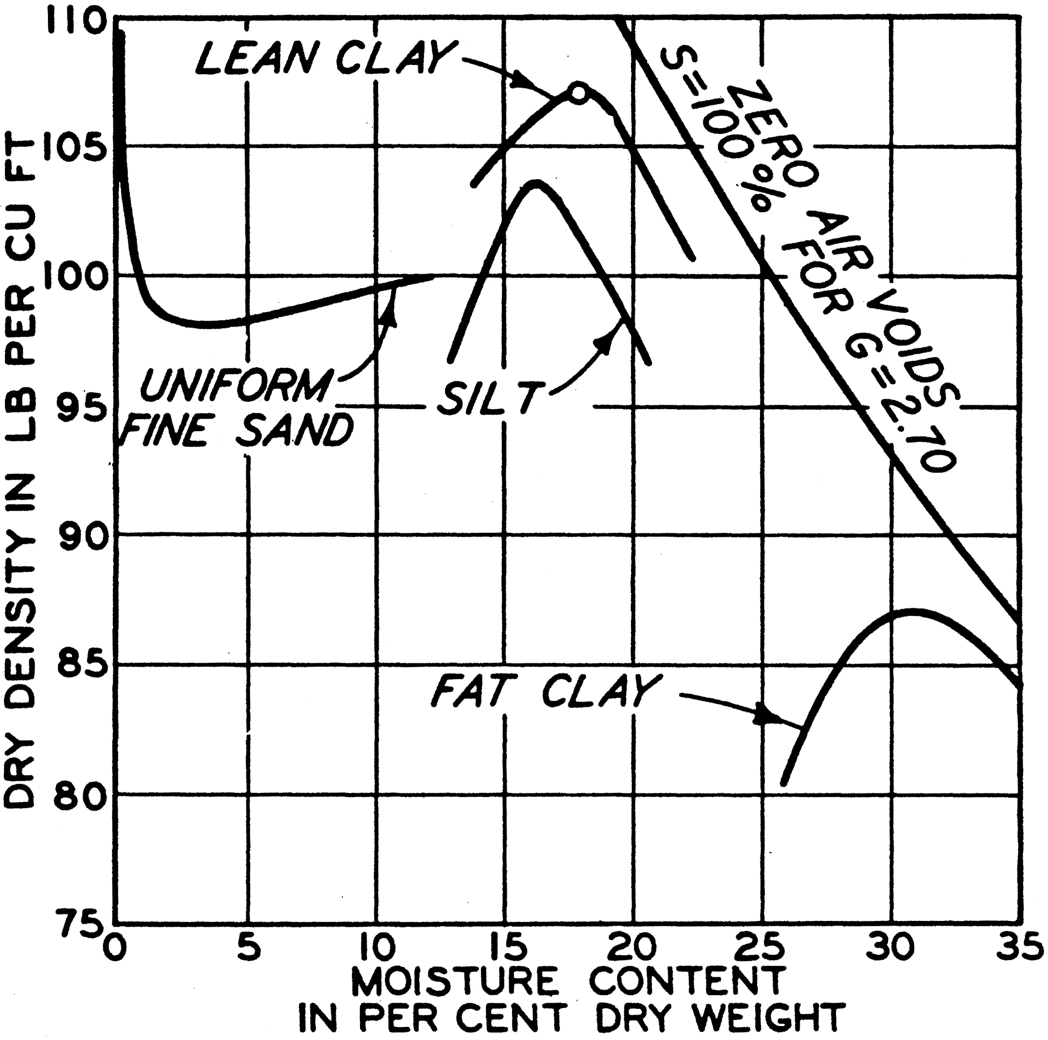
Figure 51. Unified Classification System (ASTM D-2487).

TABLE VII

CORRELATION BETWEEN EMBANKMENT PROPERTIES AND SOIL CLASSIFICATION GROUPS

GROUP SYMBOL	RELATIVE PERMEABILITY	RANGE OF K FT/YR	PIPING RESISTANCE	SHEAR STRENGTH	RELATIVE WORKABILITY
GW	Pervious	1,000 to 100,000	High	Very high	Very good
GP	Pervious to very pervious	5,000 to 10,000,000	High to medium	High	Very good
GM	Semipervious	0.1 to 100	High to medium	High	Very good
GC	Impervious	0.01 to 10	Very high	High	Very good
SW	Pervious	500 to 50,000	High to medium	Very high	Very good
SP	Pervious to semipervious	50 to 500,000	Low to very low	High	Good to fair
SM	Semipervious to impervious	0.1 to 500	Medium to low	High	Good to fair
SC	Impervious	0.01 to 50	High	High	Good to fair
ML	Impervious	0.01 to 50	Very low	Medium to low	Fair
CL	Impervious	0.01 to 1.0	High	Medium	Good to fair
MH	Impervious	0.001 to 0.1	Medium to high	Low	Poor
CH	Impervious	0.0001 to 0.01	Very high	Low	Very poor

Source: (57)



Source: (2)

Figure 52. Typical Compaction Test Data.

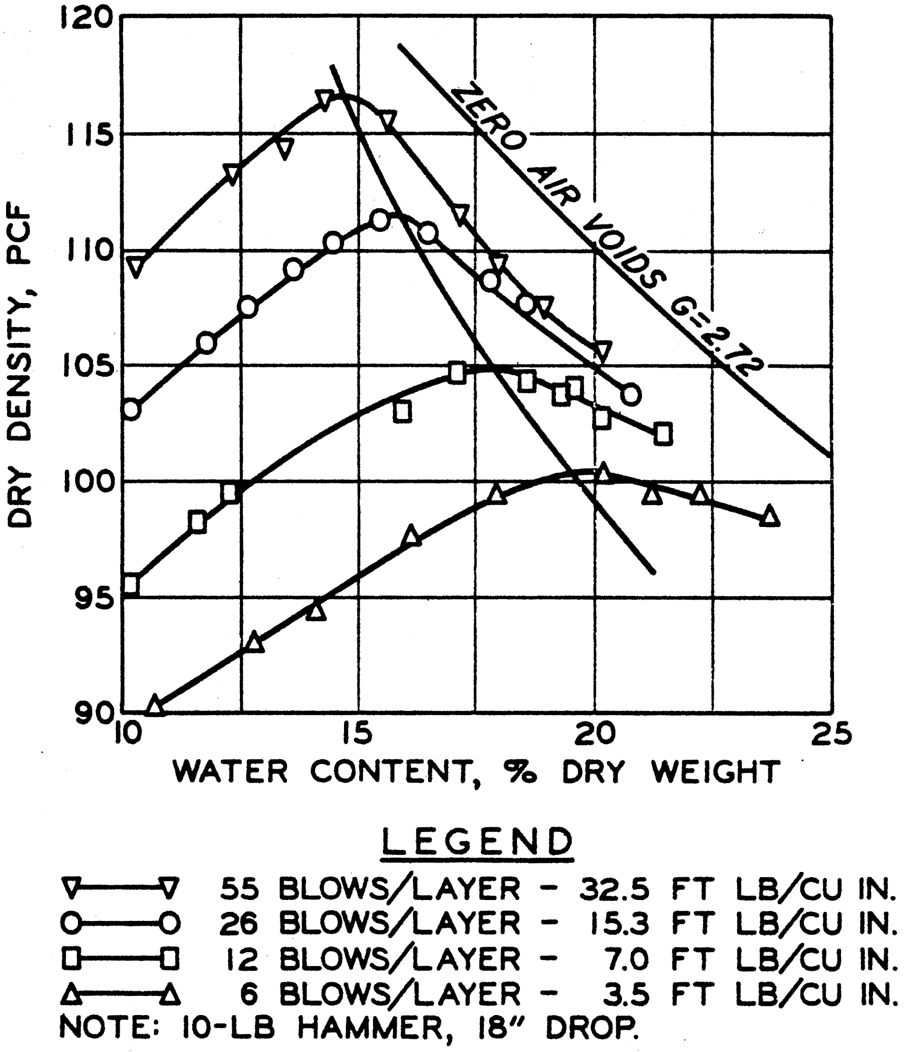
soil, which is the ratio of weight of water to weight of dry soil particles, remains essentially the same after compaction as it was before compaction. However, the degree of saturation increases because of the decrease in void space through expulsion of air with no significant decrease in water. Expulsion of all air cannot be achieved by compaction of most soils, and 100 percent saturation is not attained. For any compactive effect, an optimum water content exists which will result in the greatest dry density or state of compaction for a cohesive soil. Figure 53 presents four compaction curves relative to four compactive efforts. The optimum water content corresponds to the peak of each curve and to the maximum dry density.

Compaction of Cohesive Soils

The material properties of cohesive soil may be altered through compaction in order to enhance or modify those properties. The material properties of concern during design and construction of an earth dam are shear strength, stress-strain characteristics, compressibility, permeability, shrink-swell characteristics, and pore pressures.

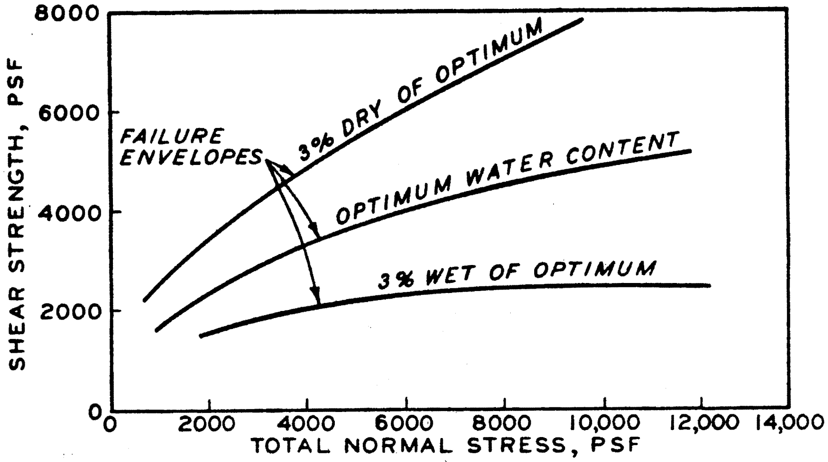
Shear Strength. The in-place shear strength of fill material is obviously a significant consideration during the construction and subsequent operational phase of an earth dam. During design, laboratory testing of remolded borrow samples can provide insight into the variations of shear strength relative to compaction water content in order to develop the most economical design to provide a safe structure.

An appreciable decrease in shear strength occurs with increasing compaction water content, as shown in Figure 54. This decrease in

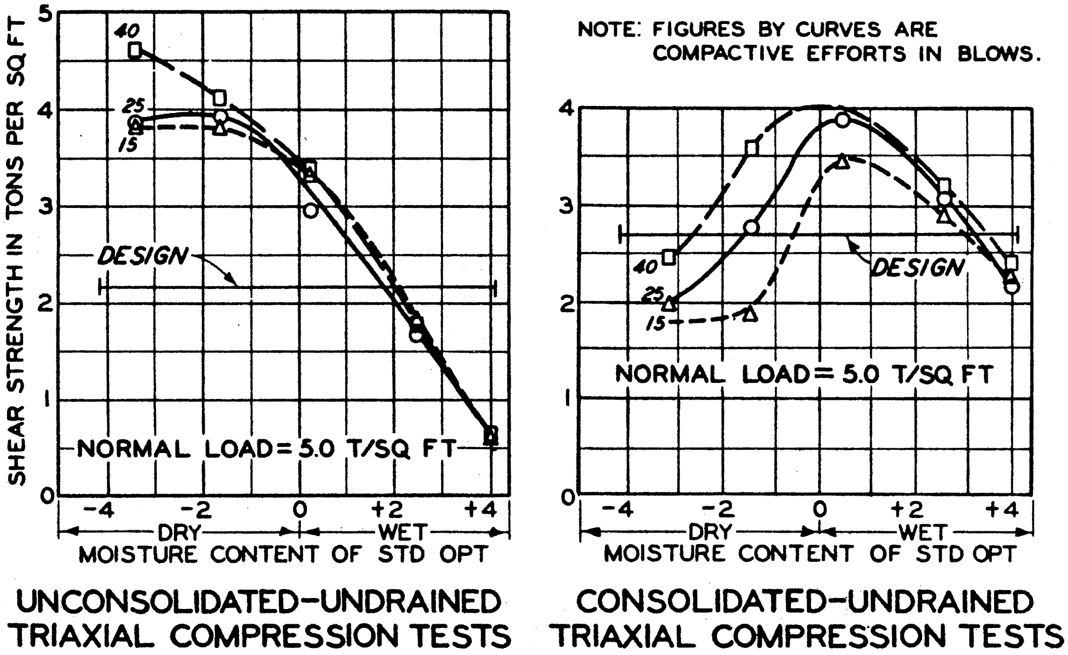


Source: (2)

Figure 53. Compaction Water Content Verses Dry Density for Low Plasticity Clay.



(a) Failure Envelopes of Low Plasticity Clay.



(b) Shear Strength Verses Water Content for Clayey Sand.

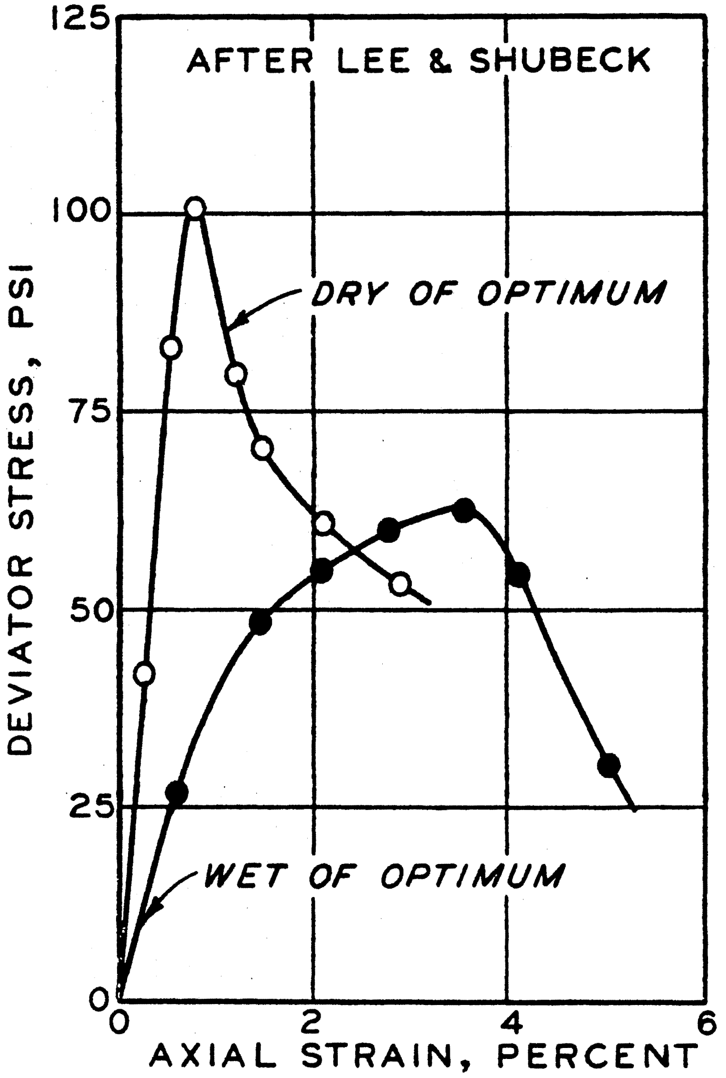
Source: (2)

Figure 54. Shear Strength as Related to Compaction Water Content.

shear strength is attributed initially to a decrease in effective stress caused by capillarity within the soil (27). As the water content is increased beyond the optimum water content, pore pressures increase due to occlusion of the air paths in the soil mass (4). The unconsolidated-undrained shear strength of the compacted soil quickly becomes directly related to the increase in pore pressures with increasing water content, as shown in Figure 54. The consolidated-undrained shear strength, independent of pore pressure influence, increases with increasing water content to optimum because of the increasing density of the soil. Beyond optimum, the shear strength decreases with decreasing density.

Zoned embankments, constructed from a single borrow area and essentially homogeneous, provide an excellent illustration of design concepts related to modification of shear strength through compaction. The central cores of these embankments are compacted slightly wet of optimum to provide a plastic membrane to resist cracking, while the outer shells are compacted slightly drier to provide greater stability through increased shear strength.

Stress-Strain Characteristics. The plastic nature of an impervious core constructed wet of optimum is readily apparent, relative to the brittle nature of outer shells compacted dry of optimum, in Figure 55. While the ultimate shear strength of soils compacted wet of optimum is somewhat lower than compacted dry, the wetter material will strain more before cracking or failing. Therein lies the importance of compacting impervious cores wet of optimum, both for static conditions and for dynamic conditions.



Source: (2)

Figure 55. Stress-Strain Characteristics of Compacted Low Plasticity Clay.

Compressibility. The compressibility of a fill material is an important parameter after completion of the embankment and particularly after impoundment of a reservoir pool. Considerable consolidation of material due to embankment loading can result in significant loss of freeboard and interior cracking. Compressibility is less related to compaction density than to compaction water content (2). Figure 56 presents consolidation test data corresponding to compaction at various water contents and subjected to different loads. The least compressible specimens are compacted from one to three percent dry of optimum. The greatest compressibilities result in specimens compacted very dry of optimum because of collapse of the structure due to wetting during the condolidation test (27). The compaction curves in Figure 56 identify problems related to interior cracking of plastic cores in areas adjacent to shells compacted slightly dry of optimum. The greater compressibility of the plastic core material may result in an appreciable differential settlement relative to the shells and interior cracking.

Permeability. The permeability of a low-plasticity clay compacted dry of optimum can be 100 times as great as the same soil compacted wet of optimum. Figure 57 presents permeabilities of a soil compacted at varying water contents and with different compactive efforts. Permeabilities of specimens compacted at equal water contents but with different compactive efforts show a decrease in permeability with increasing density. Increasing density results in decreasing void spaces, or narrowing of seepage paths. Therefore, in addition to providing a plastic core, compaction of impervious core material wet of optimum provides a more effective seepage barrier.

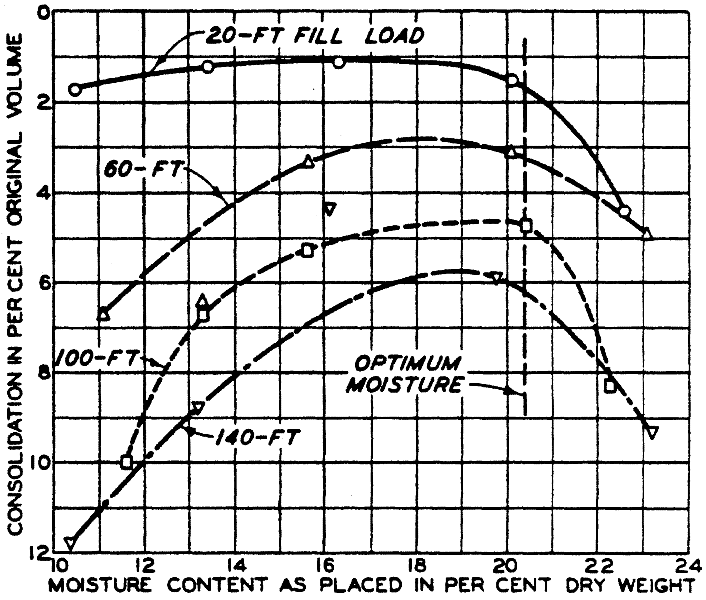


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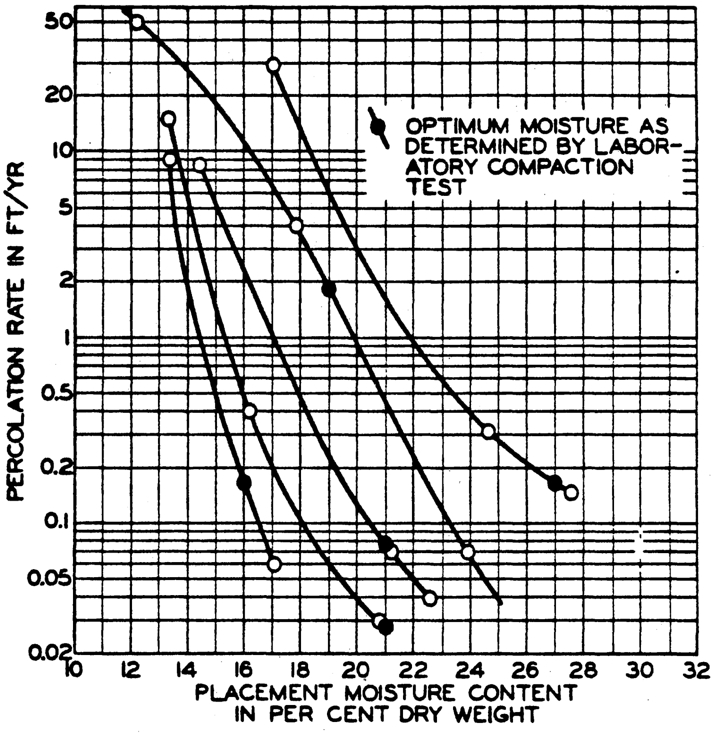
Figure 56. Effect of Water Content on Compressibility of Compacted Low Plasticity Clay.



Source: (2)

Figure 57. Effect of Water Content on Permeability of Compacted Low Plasticity Clay.

Shrink-Swell Characteristics. The potential for shrinkage of a compacted soil increases as the water content increases, while the potential for swelling decreases as the water content increases. The shrink-swell characteristics of a low-plasticity clay are shown in Figure 58.

Pore Pressures. As the compaction water content is increased beyond optimum, pore air becomes discontinuous and is no longer expulsed during compaction. As the air becomes occlusive bubbles, the compacted soil, although not saturated, will exhibit the shear strength of a totally saturated soil (219). The policy of the Bureau of Reclamation is to compact soils dry of optimum in order to avoid building in problems associated with increasing pore pressures (27). The relationship between increasing pore pressures and increasing compaction water contents is readily apparent in Figure 59.

Compaction of Cohesionless Soils

Cohesionless soils are compacted in order to simply increase their density. As the density of cohesionless soil is increased, the individual soil particles are forced closer together, thereby decreasing the compressibility of the soil and increasing the shear strength through increasing the shearing resistance. The permeability of cohesionless soils is not reduced to significant levels to be a consideration for compaction. With high coefficients of permeability, and the inherent characteristic of cohesionless soils not to retain water, pore pressures and shrink-swell effects are not factors to be considered for cohesionless soils.

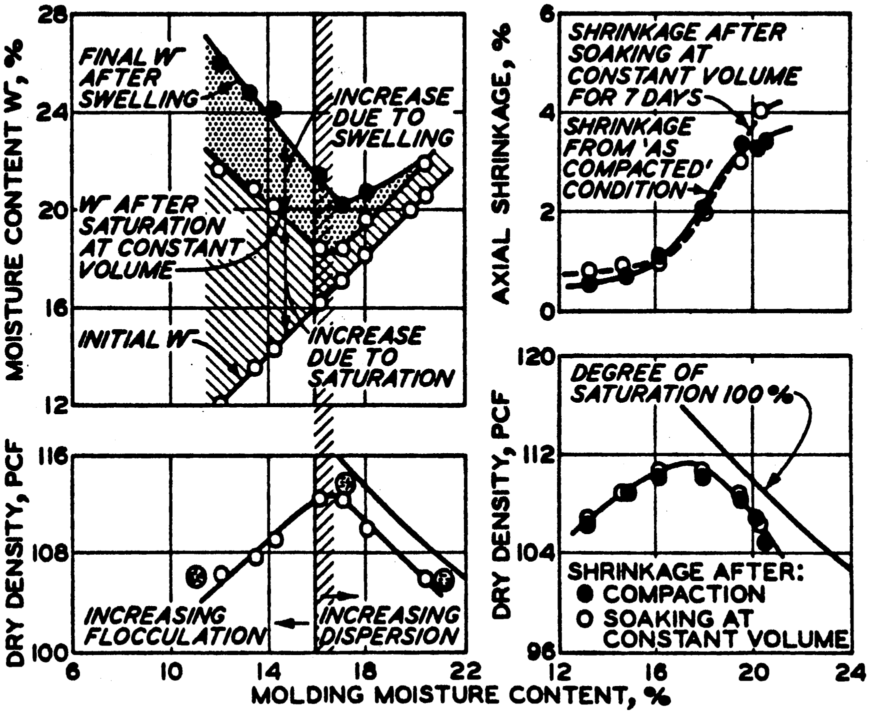


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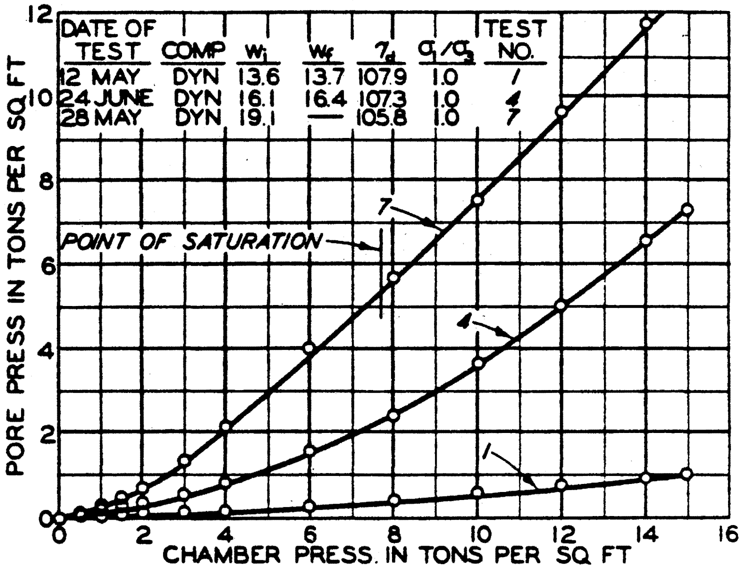
Figure 58. Shrink-Swell Characteristics of Compacted Low Plasticity Clay.



DATE OF TEST	COMP	w_i	w_f	γ_d	σ_1/σ_3	TEST NO.
12 MAY	DYN	13.6	13.7	107.9	1.0	1
24 JUNE	DYN	16.1	16.4	107.3	1.0	4
28 MAY	DYN	19.1	—	105.8	1.0	7

Source: (2)

Figure 59. Pore Pressure Relative to Compaction Water Content.

Rockfill

The properties of rockfill are dependent on the method of construction as lift thickness, compaction, and sluicing; strength, shape, and size of individual rocks; and presence of small rocks and fines incorporated into the fill. Rockfill embankments can be constructed of almost any type of rock, from uniform, large, hard rocks to soft, highly weathered rocks in which individual fragments are broken up by construction equipment and compacted like soil. Because of limitations with laboratory equipment, very little information is available from direct laboratory measurements of the properties of rockfill. Consequently, an assessment of the strength and compressibility of rockfill is determined from observations of the behavior of existing rockfill embankments or from test fills constructed specifically to establish design parameters.

Before 1964, rockfill dams were constructed by dumping rockfill in thick lifts with sluicing to facilitate settlement. The magnitude of post-construction consolidation of the rockfill was sufficient to cause damage to upstream membranes. Since 1964 all rockfill dams have been constructed by dumping into somewhat thinner lifts and compacting the rockfill. Post-construction settlement has subsequently been reduced to negligible ammounts (18).

Embankment Details

Crest

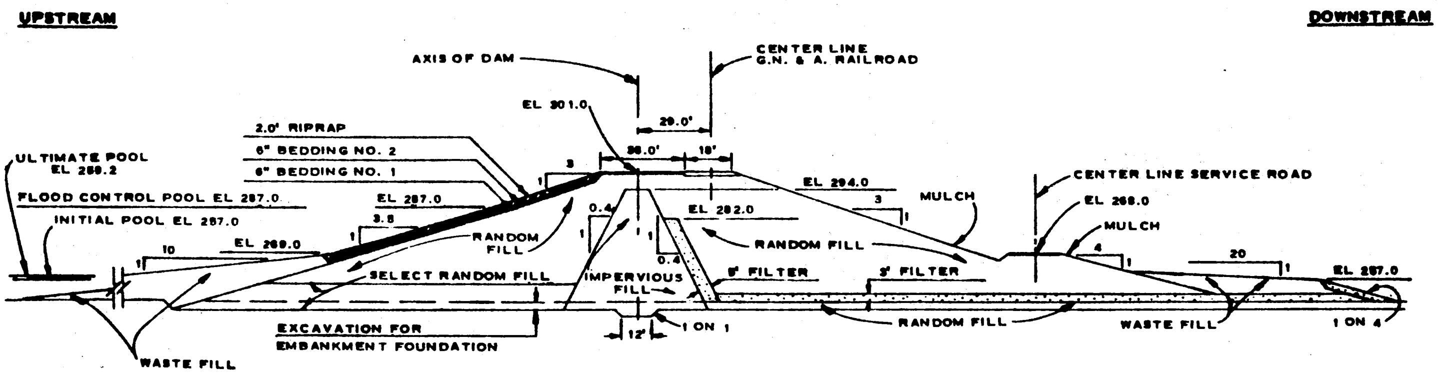
The crest width of an earth dam has no appreciable effect on embankment and foundation stability (128). Furthermore, the crest width has only a minor influence on the embankment volume (57). The crest width should, therefore, be governed by the proposed functional use. For maintenance use only, the crest width should be a minimum of 10 feet to provide vehicular access along the crest (57). For the purpose of supporting a public highway, the crest width should conform to local highway width requirements, which are typically 30 to 40 feet (128). The crest width at Millwood Dam in Arkansas is 54 feet wide to accomodate a public highway and a railroad as shown in Figure 60. For small earth dams without highway considerations, the crest width should allow minimum percolation distance through the embankment at normal reservoir pool level. The following formula has been suggested for determination of crest width for small earth dams (210).

$$w = \frac{z}{5} + 10 \tag{3.6}$$

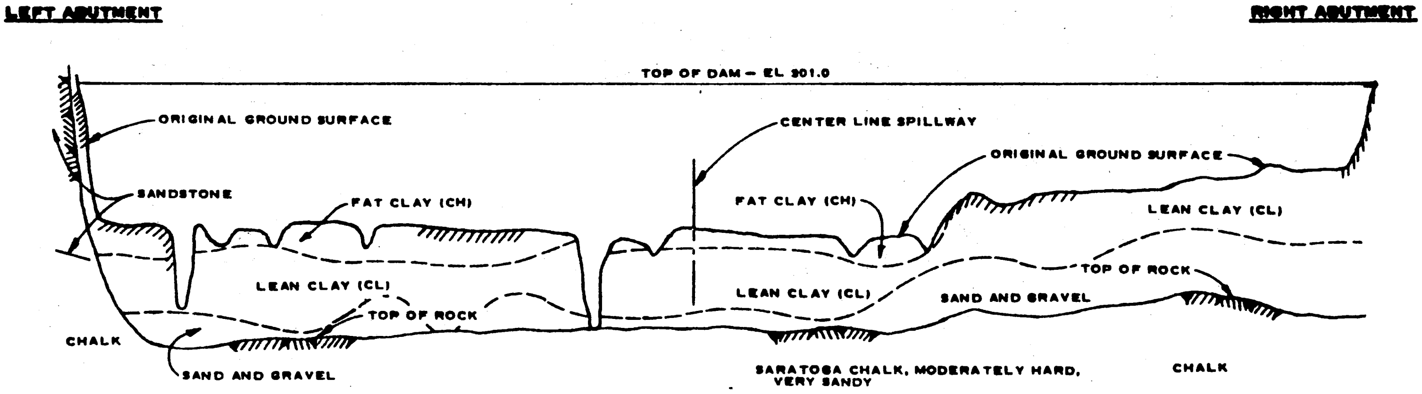
where w is the width of the crest in feet, and z is the height of the dam in feet above the streambed.

Camber, or overbuild, is usually provided along the crest to insure minimum freeboard requirements in the event of embankment settlement. Settlement of the foundation may be estimated through theoretical soil mechanics. Settlement of the embankment is best estimated through empirical studies of similar embankments. Typically, settlement of embankments corresponds to 0.1 to 0.4 percent of the embankment height (2).

Surfacing should be provided along the top of dam for protection against wave splash, frost action, storm runoff, and traffic wear. For earth dams with restricted access onto the crest, a four-inch, minimum, thickness of select, crushed rock is sufficient (210). The pavement



TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (172)

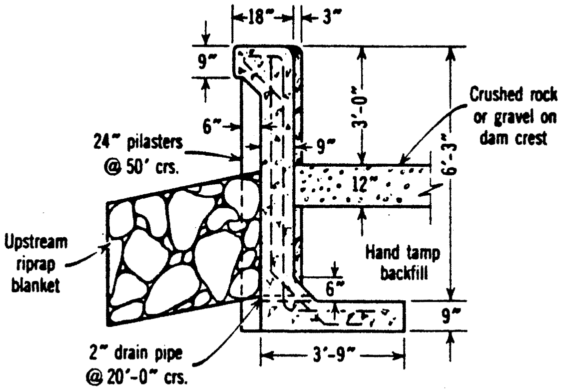
Figure 60. Millwood Dam, Little River, Arkansas.

structure of dams with public highways provide excellent protection. The crest should be sloped to drain toward the upstream slope. The upstream slope protection is in-place immediately after construction and is readily resistant to sheet flow, while considerable time is usually required to establish adequate downstream slope protection.

Freeboard

Freeboard must be provided to prevent overtopping of the dam by wave action and wind setup of the reservoir. Freeboard also acts to compensate for inaccuracies in estimating camber for post-construction settlement of the embankment and in predicting maximum flood. Normal freeboard is the vertical distance from the crest, without camber, to the spillway crest, which delineates the maximum reservoir pool. Normal freeboard must meet the requirements for long term storage to prevent seepage through a core which may be loosened by frost action or cracked through desiccation. Minimum freeboard is the vertical distance to the surface of a surcharge pool which may occur in addition to the maximum reservoir pool as a result of the maximum river flood. Minimum freeboard is required to prevent overtopping by wave action and wind setup but is not required to compensate for longterm seepage through the core (210).

Until about 1940, the Bureau of Reclamation routinely constructed concrete parapet walls along the upstream edge of the crest, as shown in Figure 61. The parapet walls were not considered in determining freeboard requirements (210). The practice was discontinued because of maintenance considerations; the walls were hindrances to snow removal from highways atop the dams (57).



Source: (211)

Figure 61. Parapet Wall as Constructed on Early Dams of the Bureau of Reclamation.

Alignment

Dam embankments which are long relative to their heights may be constructed along a straight alignment, or along a more economical alignment which better fits site topography or foundation conditions (128). Optima Dam in Oklahoma has a long and low embankment that was constructed along a straight alignment through the broad valley but along an upstream curve at the left abutment in order to avoid bentonite seams (179).

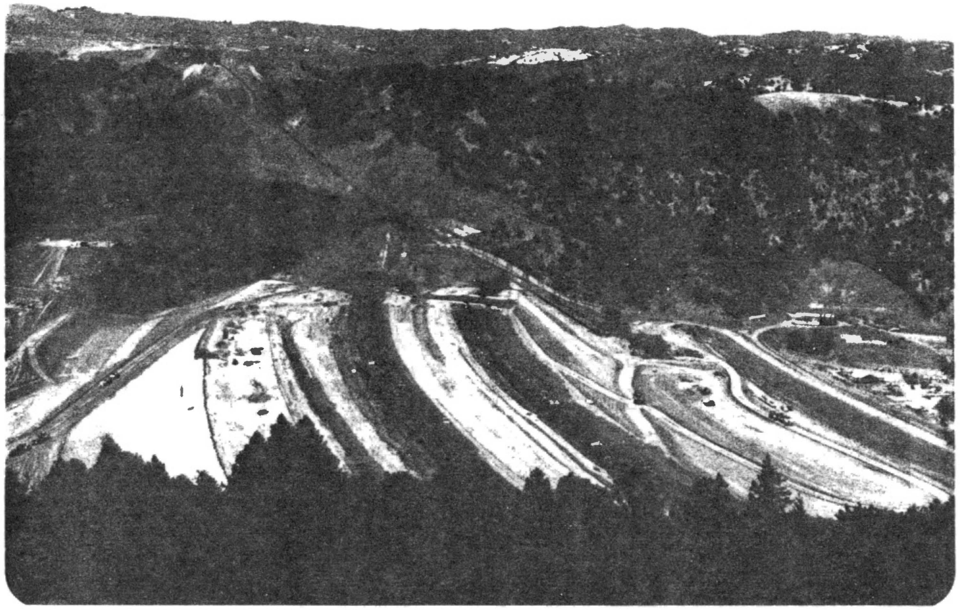
Alignments arching downstream should be avoided for all embankments. Downstream deformation of the embankments would tend to produce tension zones in the downstream shells and downstream portions of central cores, which could result in concentrations of seepage and cracking (128). Serre-Poncon Dam in France was constructed with a slight downstream arch; however, settlement in the foundation was minor and longitudinal compression was developed along the crest (57).

The axes of high dams in narrow, steep-sided valleys constructed by the Corps of Engineers are curved upstream in order that downstream deflection during reservoir loading will tend to compress the impervious zones longitudinally. This compression provides additional protection against the formation of transverse cracks within the impervious zone (128). The arching of Warm Springs Dam is shown in Figure 62. The Bureau of Reclamation is less adamant about upstream arching of dam alignments as a general policy and have evaluated each dam on a case-by-case situation (211).

Upstream arching of dam embankments is provided to produce axial compression in the core as the embankment settles. The first earth dams to be arched upstream to provide compression in the impervious core were the thin-core earth-rock dams constructed by the Aluminum Company of America (57).

Many prominent dam engineers oppose arching dams, except for thin-core dams with considerable expected settlement, because of the cost of additional surveying (59). Other prominent dam engineers, although somewhat skeptical of the benefits, argue that the additional surveying is not a significant cost item and that dam embankments should be arched in the event of obtaining possible benefits (57).

Finite element studies of the effect of reservoir filling on embankment deflections at New Melones Dam predict that the crest will deflect approximately 22 inches upstream as the reservoir pool beings to rise and will then deflect 25 inches downstream for a net downstream deflection of 3 inches when the reservoir reaches maximum pool eleva-



Source: (123)

Figure 62. Arching of Alignment of Warm Springs Dam, California.

tion (122). To date, the reservoir pool has not reached sufficient elevation to dispute or verify these predicted deflections (175). Very deep reservoir pools behind high dams may cause sufficient rotation of the foundation to produce upstream deflection during normal pool operations which could create adverse tension zones within the impervious core. This phenomenon could be similar to what is presently expected to occur as a result of downstream arching of the embankment.

CHAPTER IV

TREATMENT OF PERVIOUS FOUNDATION SOILS

General Design Considerations

Pervious foundation soils consist of pervious sands and gravels as recent alluvial deposits overlying impervious geological formations. Due to the nature of the deposition, the soils are usually stratified, resulting in considerable vertical variation and minor horizontal variation, and are best described as being a stratified heterogenous mixture.

The two concerns presented by pervious foundation soils during the design of an earth dam are (1) the quantity of potential underseepage and (2) the forces exerted by the underseepage. The first concern, that of seepage quantity, is an economic consideration. Cost studies can determine whether or not the amortized cost of the construction of a positive cutoff to eliminate seepage loss is offset by the potential economic value of the seepage loss. For example, hydropower was a proposed project purpose of Clearwater Dam in Missouri during the preliminary design phase. After detailed foundation explorations revealed substantial solutioning within the foundation limestone, hydropower was deleted as a project purpose because of the substantial expense of reducing seepage losses to negligible quantities. Clearwater Dam was completed in 1949 and has been operated continuously and successfully with 8.5 cubic feet per second seepage loss (65).

The second concern, that of seepage forces, is a geotechnical consideration as related to the safety of the embankment structure. Of the potential failure modes relative to earth dams, piping failure by subsurface erosion within undisturbed foundation materials is the only failure mode which cannot be resisted directly by design and construction. Resistance to piping in the foundation can only be achieved by the design and construction of drainage provisions which prevent seepage removal of particles from the foundation substrata.

The treatment of pervious foundation soils must address both concerns. The treatment must be compatible with the purpose of the dam and the economic value of seepage losses and the treatment must result in a safe structure. Three basic methods of treatment of pervious foundation soils are possible (57): (1) eliminating the seepage or reducing it to a negigible amount by constructing a vertical foundation seepage barrier; (2) reducing the seepage, either with a partial cutoff or with an upstream impervious blanket, and providing for the control of the reduced seepage; or (3) taking no steps to reduce the seepage, and providing for its control.

If the construction of a positive cutoff is not a significant cost item for a dam project, the cutoff should be included. The positive cutoff would provide a substantial reduction in potentially hazardous underseepage in a quality consistent with the embankment, particularly if the cutoff is constructed of rolled earthfill. An additional advantage of a rolled-earth cutoff is that the trench excavation allows visual inspection of the insitu foundation soils and of the top of rock. The rolled-earth cutoff under El Dorado Dam in Kansas was included early in the construction phase because of minor added expense, although exploration during design did not reveal a need for a positive cutoff. During excavation of the cutoff trench the underlying foundation limestone was found to have undergone excessive solutioning. Whether a positive cutoff is deemed necessary for satisfactory operation of the project or added because of a minor additional cost, the safety of the structure should not be construed as greater than without the cutoff. Inspections along with monitoring of installed instrumentation should routinely continue in order to insure the safety of the dam.

Methods for Preventing or Reducing Underseepage

Treatment of pervious foundation soils to eliminate underseepage below a dam involves the construction of a vertical impervious barrier for the full depth of the pervious foundation soils. Generally the impervious cutoff is founded in rock, but satisfactory performance has been achieved by founding the cutoff in an impervious soil stratum. Cutoffs have been constructed using rolled impervious earthfill over rock, rolled impervious earthfill in excavated trenches, steel sheet piling, concrete walls, slurry trenches and alluvial grouting.

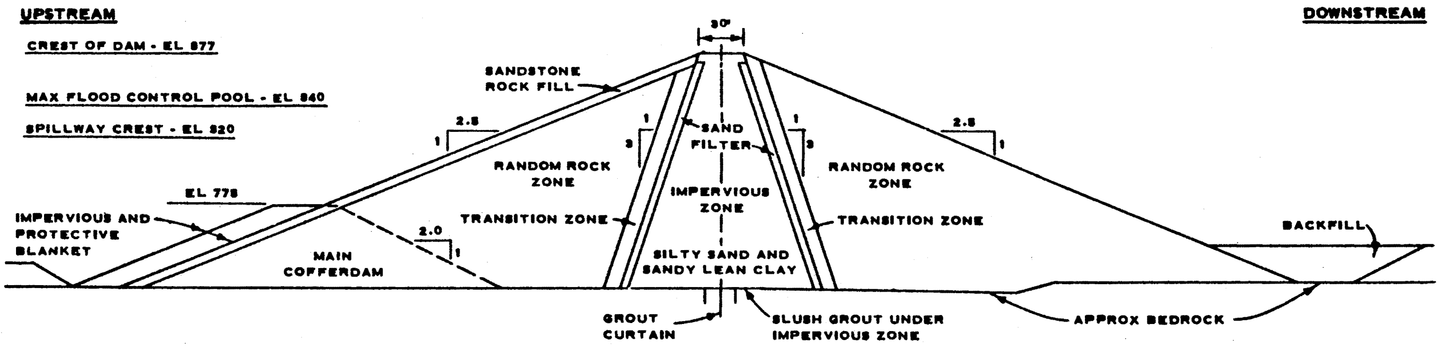
Removal of Pervious Soil

The considerations for removal of pervious foundation soil are stability of the embankment and access for treatment of foundation rock, in addition to preventing underseepage. Dam sites where complete removal of foundation soils has been accomplished had contained only minor amounts of soils. These soils were of shallow depth or of small areal extent, or both. With the foundation soil removed, treatment to eliminate underseepage involved only the foundation rock.

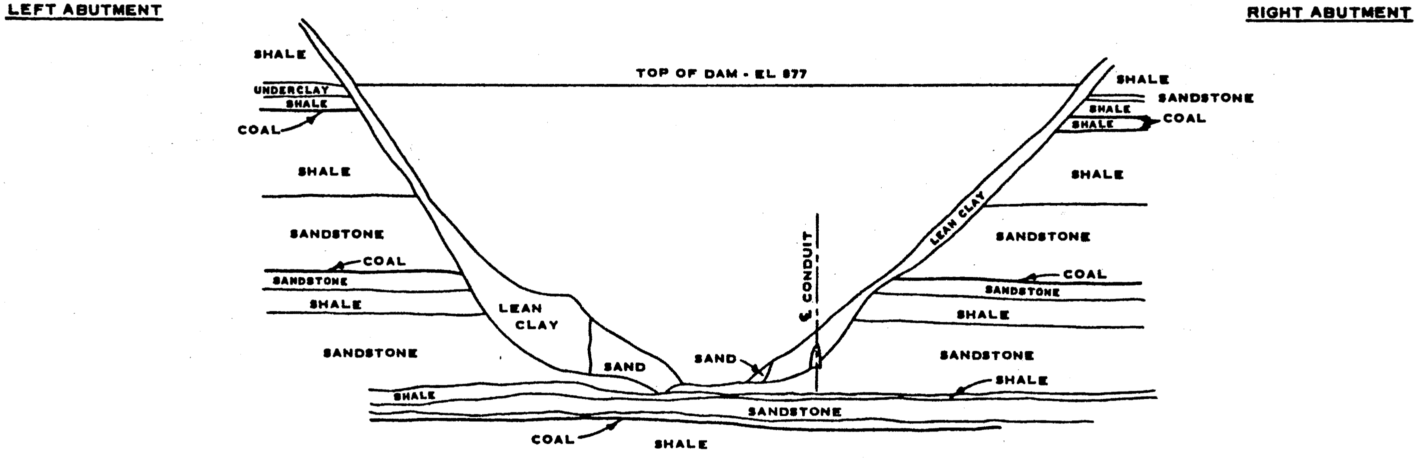
For the construction of Buckhorn Dam in Kentucky, all overburden was removed from the embankment foundation and stockpiled for construction of the impervious core of the embankment. The only treatment of the foundation rock consisted of grouting operations beneath the impervious core (75). The typical embankment section and geologic cross section of Buckhorn Dam is presented in Figure 63. A common treatment of foundation rock after removal of overburden soils is the construction of a core trench excavated to sound rock and backfilled with impervious earthfill material.

The damsite for Salamonie Dam in Indiana contained a negligible amount of alluvial soils in the valley section and deep glacial till as the abutments. Only the alluvial soils were removed prior to construction of the embankment. A cutoff trench was extended part way into the abutments (182). A typical embankment section and geologic cross section of Salamonie Dam is presented in Figure 64. Overburden removal at Nolin River Dam in Kentucky was seemingly the opposite. The damsite for Nolin River Dam was covered with a negligible amount of overburden in the abutment and up to 40 feet of impervious soils in the valley section (97). All overburden was removed except for the impervious soils immediately below the random fill zone of the embankment as may be seen in Figure 65.

Talus material should be excavated and removed from the dam foundation area prior to final foundation treatment and construction of the embankment. The boulders and rock blocks comprising the larger talus fall on recent alluvium after becoming detached from contributing



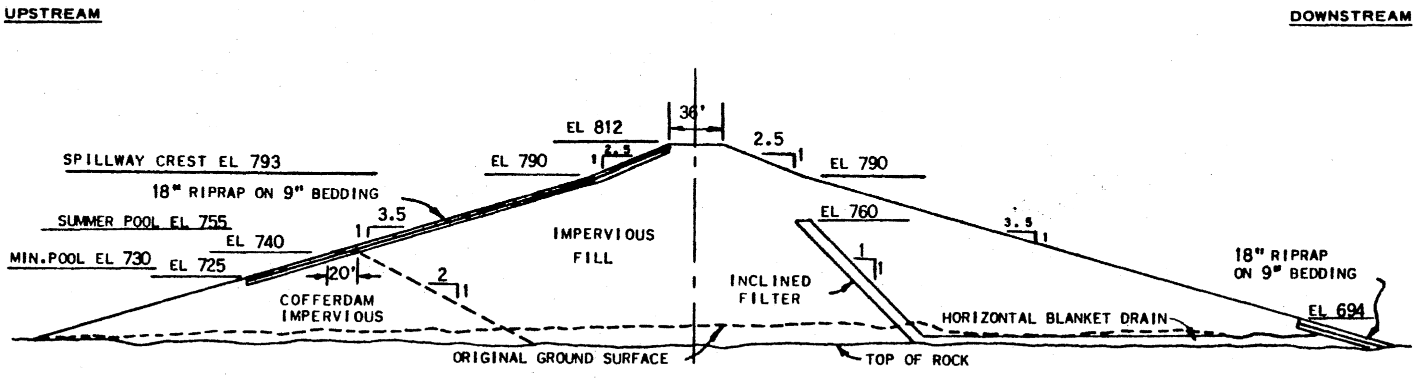
TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (75)

Figure 63. Buckhorn Dam, Middle Fork Kentucky River, Kentucky.



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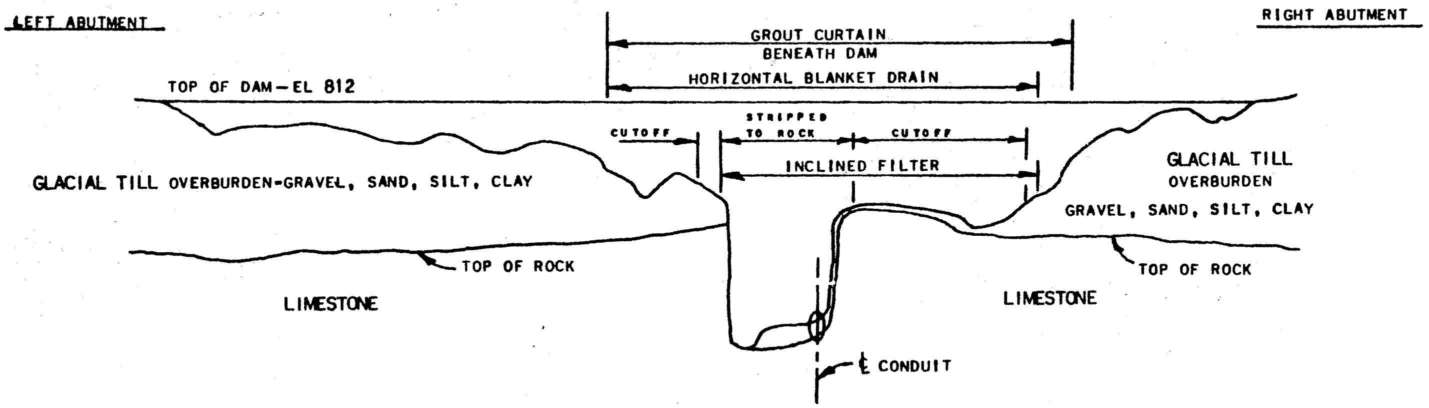
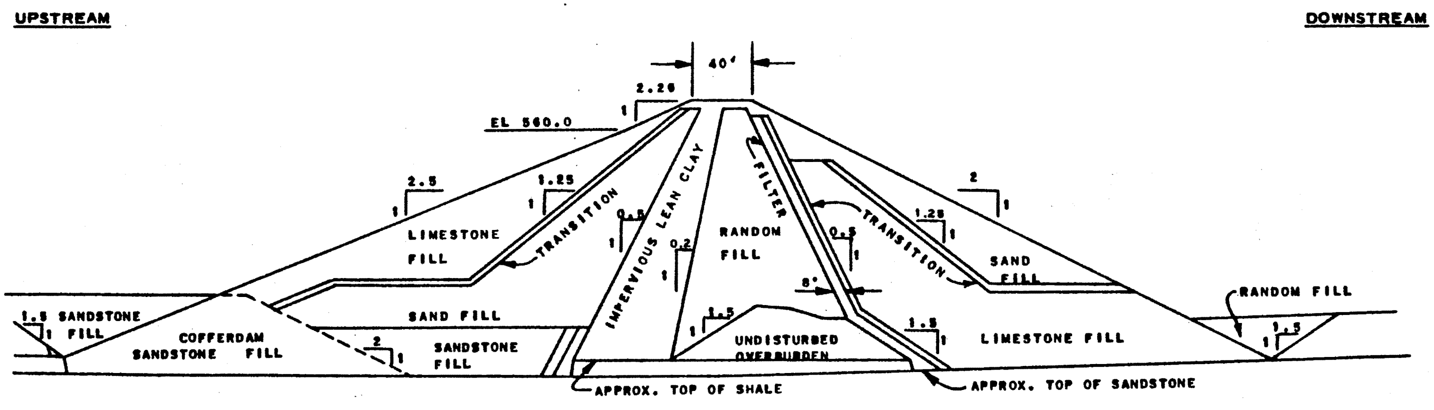


VALLEY CROSS SECTION

Source: (182)

Figure 64. Salamonie Dam, Salamonie River, Indiana.



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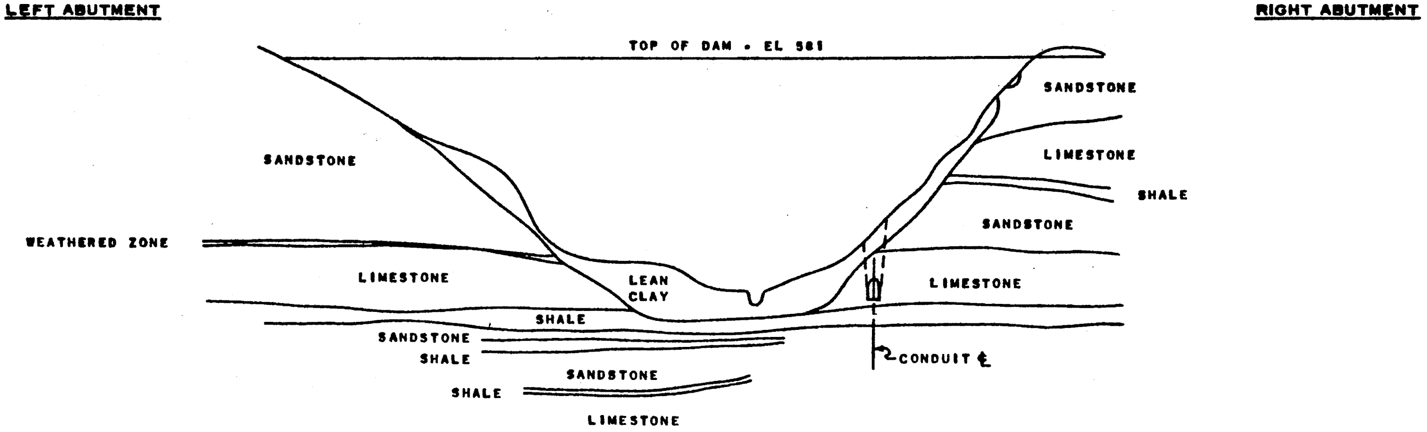


VALLEY CROSS SECTION

Source: (97)

Figure 65. Nolin River Dam, Nolin River, Kentucky.

cliffs. Large voids occur between these individual boulders and rock blocks. Large voids are also created from partial contact with underlying alluvium or are developed by stream scour or consolidation of the alluvium. In addition to stability problems which may arise as a result of leaving the talus material within the dam foundation, the voids provide the opportunity for undetectable piping within the foundation. This situation was well illustrated during foundation preparation of Everett Dam in New Hampshire. The overburden of the right abutment consisted of approximately 30 feet of talus material avalanched from the cliffs above. A major construction item regarding foundation treatment was the removal of the talus material. During excavation of the talus to the underlying rock surface, large voids were encountered between the boulders and rock blocks, as anticipated during the design phase (80).

Earthfill Cutoff Trenches

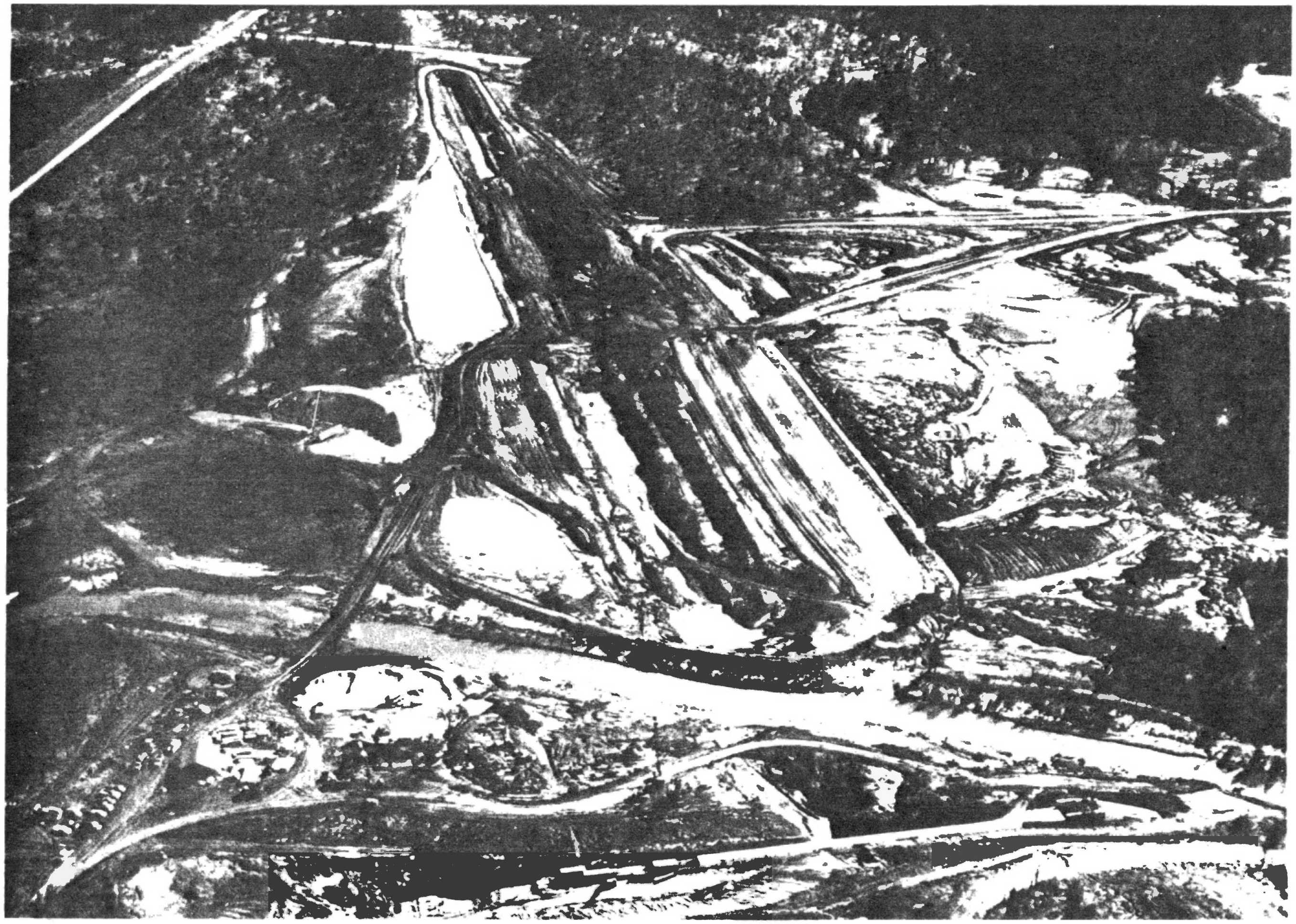
An earthfill cutoff trench, also referred to as a rolled-earth cutoff or a core trench, is an open excavation with sloping sides along the alignment of a dam embankment and backfilled with impervious soils to provide an impervious barrier to prevent underseepage through the foundation. Historically, vertical sides have not been used because of the expense of hand labor for compaction and because of the difficulties in maintaining a vertical cut. Furthermore, vertical trenches have not been used because of the realization of the potential for hydraulic fracturing of the core material near the top of the trench. The effective stresses within the core material near the top of the trench would be significantly less than the effective stresses within the overlying embankment material because of the tendency for the embankment to bridge over a narrow, vertical-sided cutoff trench. For a high dam the hydrostatic pressures within the foundation induced by high reservoir levels could be greater than the effective stress in the core trench thereby enabling the reservoir-induced pressure to hydraulically fracture the cutoff trench. To prevent hydraulic fracturing, high dams require large widths of contact between the embankment core and the cutoff trench and sloping sides for the cutoff trench.

Because of the excavation, a more concise evaluation of the dam foundation can be ascertained than by exploratory drilling and sampling during the design phase. This more concise evaluation of the foundation may result in modification of the proposed foundation treatment or in modification of the embankment design, or both. Ideally the prospect of modification should be a design consideration with allowance for rescheduling construction times and funds. As the dam foundation can be well examined within the trench excavation, so too can the construction of the impervious cutoff be well examined. The same construction methods and the same impervious material as used in the embankment core can be used in the cutoff trench. Conscientious supervision of materials and materials placement will result in an effective seepage barrier which will provide satisfactory performance. All other methods for construction of a cutoff may be defective in spite of conscientious supervision (62).

The major difficulty, and the corresponding major expense, in excavating and backfilling an earthfill cutoff trench is the dewatering of the work area. The nature of pervious foundation soils requires a wellpoint dewatering system in order to lower the groundwater elevation and to maintain stability of the side slopes of the trench excavation (14). Figure 66 shows the cutoff trench excavation of Birch Dam in Oklahoma and Figure 67 shows the installed well points along the base of that excavation. Small sump pumps can easily accommodate the groundwater flow into rock excavation or into excavations within very impervious soils. The overburden at most damsites is much too shallow to dewater with deep-well pumps. Deep-well pumps are adaptable, in conjunction with sheet piling, for dewatering very deep, very porous overburden as found below the locks and dams along the Mississippi River.

The cutoff trench should be located upstream of the axis of the dam but not beyond sufficient cover provided by the impervious core of the embankment (210). The requirement for sufficient embankment cover will cause the cutoff trench to converge toward the axis at the abutments. Locating the cutoff trench along the axis prevents uplift pressures from occurring below the downstream portion of the embankment. Locating the cutoff trench upstream of the axis prevents uplift pressures from occurring below a greater portion of the embankment. In practice, the cutoff trench is located below the impervious material of the embankment; for a dam with a central impervious core the cutoff is placed at the axis, and for a dam with an upstream impervious zone the cutoff trench is located upstream of the axis.

The bottom width of the cutoff trench should be such as to provide a satisfactory seepage barrier and adequate contact with the underlying impervious stratum. In current practice the bottom width is established as a function of the reservoir height, as is the width of the impervious embankment zone. The Bureau of Reclamation recommends that



Source: (5)

Figure 66. Cutoff Trench Excavation at Birch Dam.

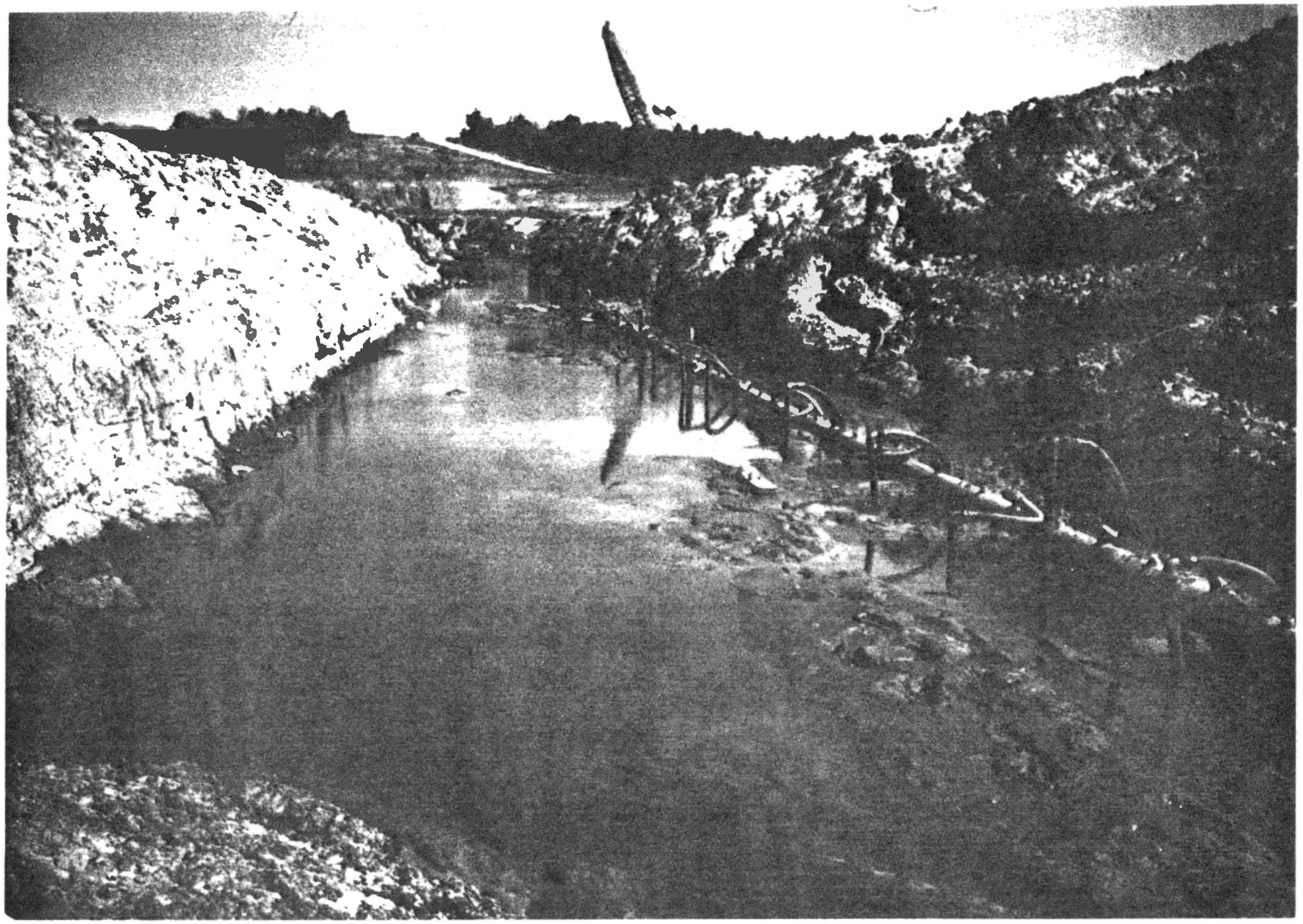


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Figure 67. Wellpoint Dewatering System along Base of Cutoff Trench Excavation.

the bottom width of the cutoff trench be equal to the reservoir head above ground surface minus the depth of the cutoff trench below ground surface. The rationale for subtracting the depth of the cutoff trench is that the seepage force at the rock contact will decrease due to head loss in traveling vertically through the foundation (210). This rationale is valid only for partial cutoffs founded in a stratum less impervious than the cutoff trench material. Within the foundation of most earth dams the flow lines are nearly horizontal. Consequently the equipotential lines are nearly vertical indicating no significant decreases in seepage pressures with a decrease in depth within the foundation. The Corps of Engineers recommends that the bottom width be equal to one-fourth of the maximum reservoir head (128). The minimum bottom width for any cutoff trench is 20 feet to allow room for processing equipment. The 10-foot bottom width of the cutoff trench at Wolf Creek Dam in Kentucky was criticized during seepage investigations for having impeded proper compaction of the impervious material and is considered a factor in the development of underseepage through the foundation (28). Table VIII presents a selection of earth and rockfill dams which have cutoff trenches for the full length of the embankment.

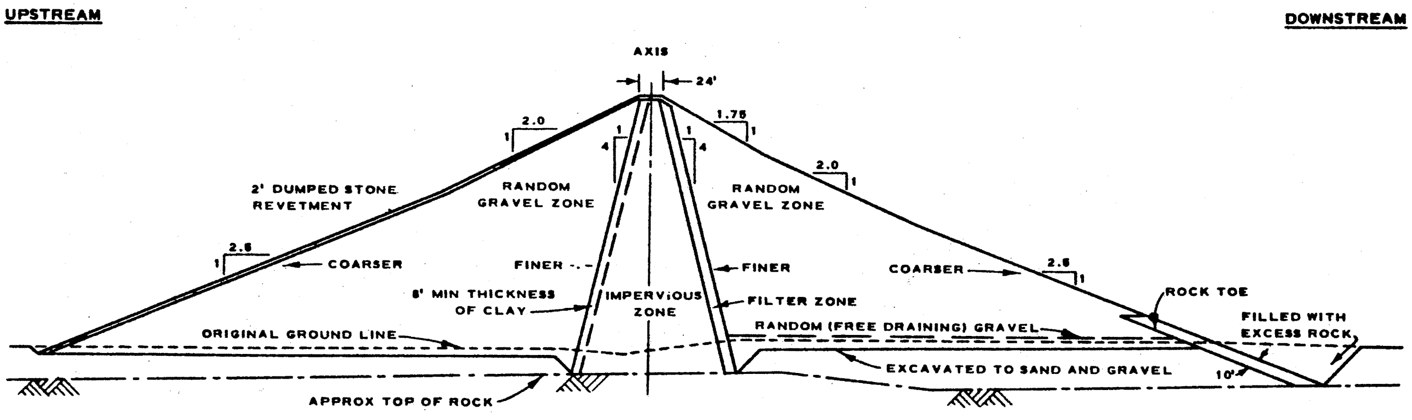
Current practice in the construction of cutoff trenches for high dams is to provide a sufficiently wide excavation to accommodate the full width of the impervious core of the embankment. This trend is well illustrated for central core embankments by the typical section of Lookout Point Dam in Oregon in Figure 68. At Abiquiu Dam in New Mexico the width of the top of the cutoff trench equals the bottom width of the impervious embankment core, as shown in Figure 69.

The practice of removing sufficient foundation soils in order to

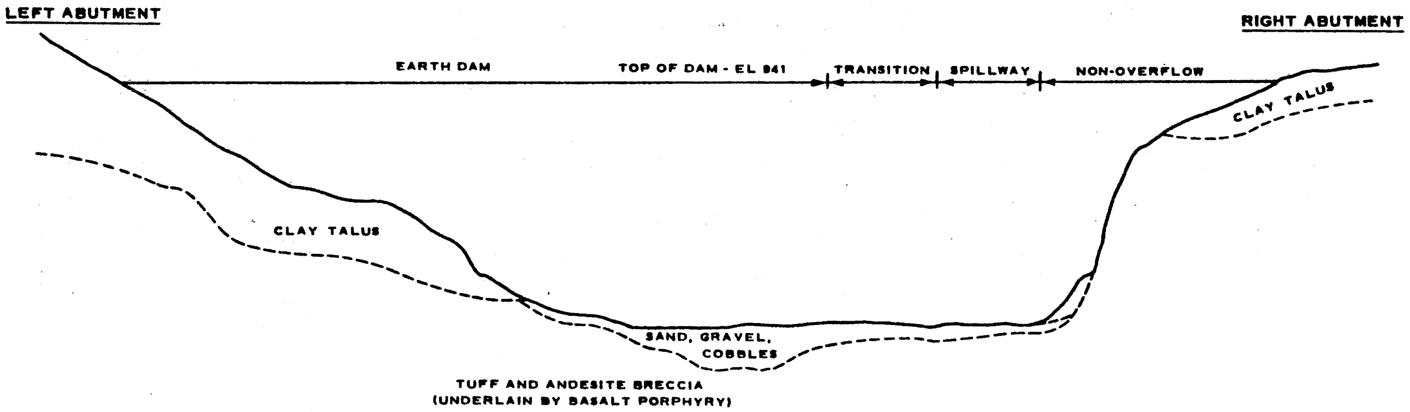
TABLE VIII

PARTIAL LISTING OF EARTH DAMS WITH CUTOFF TRENCHES FOR FULL LENGTH OF THE EMBANKMENT

Name of Dam		H	D	EMB IMP	Location	Cutoff Trench		Slopes	Founded	Rock Type	Reference
						Width	Depth				
Abiquiu, New Mexico	1963	320	302	Central	Axis	core	30	1V-on-1.5H	1 ft into rock	Mudstone, sandstone	(82)
Ball Mountain, Vermont	1961	265	230	Central	US	25	25	1V-on-1.5H	Top of glacial till	Glacial till	(79)
Bear Creek, California	1955	92	71	Central	Axis	core	20	1V-on-2H	Top of rock	Schist	(138)
Belton, Texas	1954	192	161	Central	US	30	45	1V-on-1.5H	Top of rock	Shale	(90)
Benbrook, Texas	1952	130	93	Homo	US	10	30	1V-on-1H	Firm rock	Limestone, shale	(199)
Birch, Oklahoma	1977	97	76	Central	Axis	25	35	1V-on-2H	Firm rock	Shale, sandstone	(140)
Black Butte, California	1963	140	99	Central	Axis	core	15	1V-on-1H	Top of rock	Sandstone, conglomerate	(95)
C.J. Brown, Ohio	1973	72	55	Central	Axis	20	25	1V-on-1.7H	1 ft into till	Glacial till	(117)
Cagles Mill, Indiana	1952	147	121	Central	Axis	15	35	1V-on-1.5H	Top of rock	Limestone	(142)
Canyon, Texas	1964	224	193	Central	Axis	20	50	1V-on-1H	Firm rock	Limestone, shale	(98)
Carlyle, Illinois	1967	67	46	Homo	US	25	40	1V-on-1.5H	Top of rock	Siltstone, shale, sandstone	(143)
Clearwater, Missouri	1948	154	113	Sloping	US	35	35	1V-on-2H	Top of rock	Limestone	(65)
Cold Brook, South Dakota	1953	130	101	Sloping	US	40	45	1V-on-1.5H	Firm rock	Limestone,sandstone	(118)
Colebrook River, Connecticut	1969	223	194	Central	Axis	40	45	1V-on-1.5H	Top of rock	Gneiss, granite	(114)
Conant Brook, Massachusetts	1966	85	71	US	US	25	30	1V-on-2H	Top of rock	Schist	(145)
Coralville, Iowa	1958	96	65	Central	Axis	15	40	1V-on-1.5H	Top of rock	Limestone	(76)
Cottonwood Springs, S. Dakota	1969	123	104	Central	Axis	30	25	1V-on-2H	Firm rock	Shale, siltstone	(112)
Cougar, Oregon	1964	445	392	Sloping	US	core	40	1V-on-1H	Top of rock	Mudstone, tuff	(92)
Coyote Valley, California	1959	164	145	Central	Axis	60	20	1V-on-1H	Top of old alluvium	Older alluvium	(70)
Crooked Creek, Pennsylvania	1940	143	115	Central	Axis	25	35	1V-on-1.5H	Firm rock	Shale	(146)
Curwensville, Pennsylvania	1965	131	102	Central	Axis	30	30	1V-on-1.5H	Firm rock	Sandstone, siltstone, shale	(100)
Deer Creek, Ohio	1968	88	44	Homo	Axis	30	35	1V-on-2H	Top of rock	Dolomite	(148)
East Branch, Connecticut	1964	92	76	Sloping	US	25	20	1V-on-1H	Top of rock	Gneiss	(151)
Eau Galle, Wisconsin	1969	122	103	Central	Axis	40	35	1V-on-2H	Firm rock	Siltstone, sandstone	(110)
El Dorado, Kansas	1981	99	83	Central	Axis	25	35	1V-on-1.5H	Firm rock	Limestone	(152)
Eufaula, Oklahoma	1964	114	67	Central	Axis	49	45	1V-on-1.5H	Firm rock	Shale	(154)
Everett, New Hampshire	1962	115	98	Central	Axis	30	30	1V-on-1H	Top of rock	Quartz	(80)
Ferrells Bridge, Texas	1958	97	70	US	US	15	35	1V-on-1H	2 ft into clays	Indurated sands and clays	(156)
Francis E. Walter, Penn.	1960	234	210	Central	Axis	20	100	1V-on-1H	Top of rock	Quartzitic sandstone	(84)
Grapevine, Texas	1952	137	109	Central	US	20	20	1V-on-1.5H	Top of rock	Shale, sandstone	(158)
Grayson, Kentucky	1967	125	96	Central	Axis	25	60	1V-on-2.5H	Firm rock	Sandstone	(159)
Hall Meadow Brook, Conn.	1962	73	54	Sloping	US	25	10	1V-on-1H	Top of rock	Gneiss, schist	(161)
Hancock Brook, Connecticut	1966	57	36	US	US	25	30	1V-on-2H	Top of rock	Schist	(162)
Harlan County, Nebraska	1952	107	69	Homo	US	25	40	1V-on-2H	Top of rock	Chalk	(200)
Hills Creek, Oregon	1961	325	273	Central	Axis	core	40	1V-on-1H	Top of rock	Andesite	(101)
John Redmond, Kansas	1964	84	36	Central	Axis	25	30	1V-on-1H	Top of rock	Shale, limestone	(169)
Kaneohe-Kailua, Hawaii	1980	76	56	Central	Axis	30	20	1V-on-2H	Firm rock	Tuff, basalt	(120)
Keystone, Oklahoma	1964	121	69	Central	Axis	25	45	1V-on-2.5H	10 ft into rock	Shale	(171)
Lewisville, Texas	1954	125	97	Homo	Axis	20	40	1V-on-2H	Top of rock	Shale	(135)
Littleville, Massachusetts	1964	164	144	US	US	40	40	1V-on-2H	Top of till	Glacial till	(96)
Lookout Point, Oregon	1954	273	220	Central	Axis	core	45	1V-on-1H	Top of rock	Tuff, breccia	(86)
Lucky Peak, Idaho	1955	340	323	Central	Axis	150	80	1V-on-2H	Top of rock	Granite, basalt	(85)
Melvern, Kansas	1973	123	102	Central	Axis	30	25	1V-on-2.5H	Firm rock	Shale, limestone	(121)
Northfield Brook, Connecticut	1964	118	103	Homo	US	35	20	1V-on-1H	Top of rock	Schist	(176)
Painted Rock, Arizona	1960	181	137	Central	Axis	76	70	1V-on-1.5H	Top of rock	Basalt, rhyolite, tuff	(73)
Sardis, Oklahoma	1982	83	63	Central	Axis	40	25	1V-on-1.5H	Firm rock	Shale, sandstone	(184)
Stillhouse Hollow, Texas	1968	200	168	Central	Axis	40	25	1V-on-2H	Firm rock	Limestone, shale	(109)
Success, California	1961	142	103	Central	Axis	core	57	1V-on-1H	Top of old alluvium	Older alluvium	(94)
Tenkiller Ferry, Oklahoma	1952	197	162	Central	US	40	40	1V-on-1H	10 ft into rock	Sandstone, shale	(186)
Thomaston, Connecticut	1960	142	119	Sloping	US	20	60	1V-on-1H	Top of rock	Granite	(78)
Tionesta, Pennsylvania	1941	154	125	Central	Axis	25	50	1V-on-1.5H	Top of rock	Shale, sandstone	(187)
W. Kerr Scott, North Carolina	1968	148	116	Sloping	US	20	20	1V-on-2H	Firm rock	Schist	(88)
Whitlow Ranch, Arizona	1960	149	116	Central	Axis	core	40	1V-on-1.5H	Top of rock	Rhyolite	(192)
Whitney, Texas	1951	159	112	Homo	Axis	20	40	1V-on-1.5H	Firm rock	Limestone	(193)
Wolf Creek, Kentucky	1950	185	135	Homo	US	10	70	1V-on-1.5H	Firm rock	Limestone	(28)



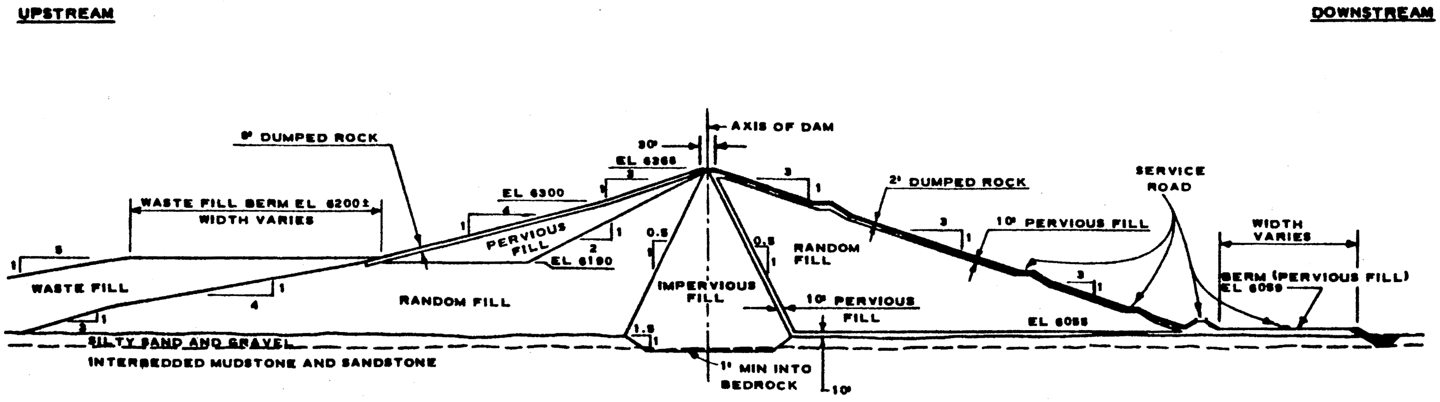
TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (86)

Figure 68. Lookout Point Dam, Middle Fork Willamette River, Oregon.



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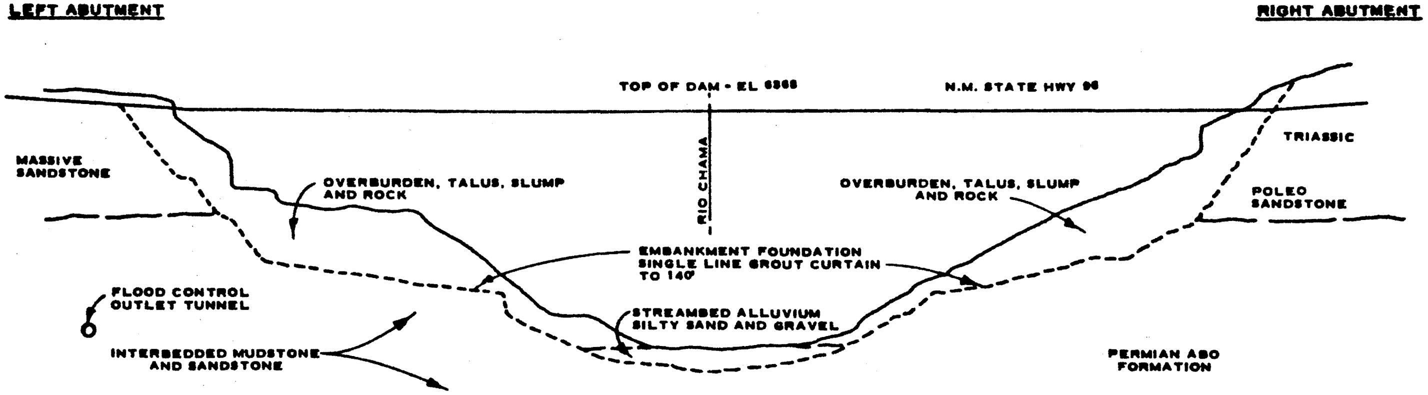


VALLEY CROSS SECTION

Source: (82)

Figure 69. Abiquiu Dam, Rio Chama, New Mexico.

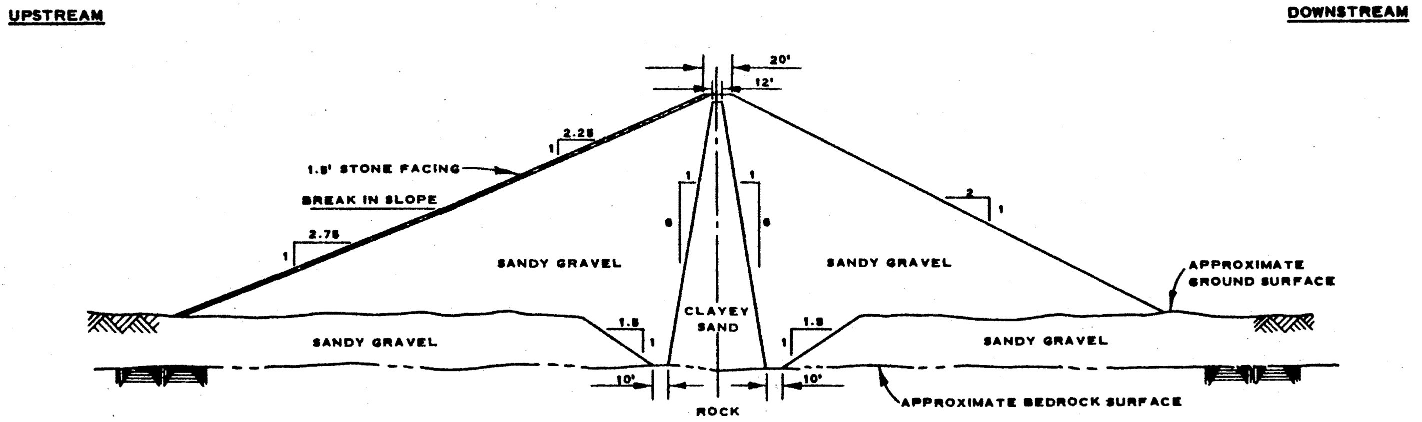
extend the impervious embankment core to rock has had some application at dams of intermediate heights such as Whitlow Ranch Dam in Arizona, Figure 70. However, the more standard approach has been to designate a bottom width somewhat less than the width of the impervious embankment core. This approach is represented by the 25-foot bottom width at Keystone Dam in Oklahoma, Figure 71, and by the 40-foot bottom width at Littleville Dam in Massachusetts, Figure 72.

For small dams, the minimum width as required for processing equipment is taken as the bottom width of the cutoff trench. The bottom widths for the homogeneous embankment of Carlyle Dam in Illinois, Figure 73, and the zoned embankment of Hancock Brook Dam in Connecticut, Figure 74, were established at 25 feet.

A cutoff trench through poorly graded, coarse foundation soils or into highly fractured rock requires a filter to prevent migration of the fine-grained cutoff material into the foundation soils or fractured rock. The design of this filter follows the same criteria as for the internal filters of the embankment. The configuration and extent of the filter is site specific and may be required for the full depth of the cutoff trench or may be limited to placement over a particularly pervious stratum. The trench filter may be connected to the internal filters of the embankment for discharge at the toe, as for Wilson Dam in Kansas which is presented in Figure 75, or may be isolated as at Cold Brook Dam in South Dakota as shown in Figure 76.

In general, partial cutoff trenches have been shown to be ineffective in significantly reducing underseepage through a dam foundation (42). The results of these studies are plotted in Figure 77.

Exceptions to the ineffectiveness of partial cutoffs include



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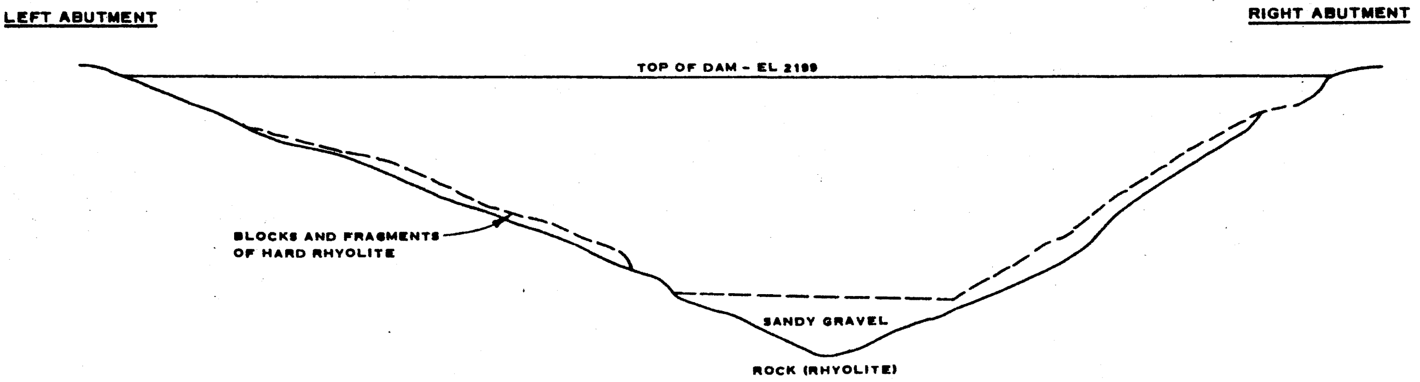
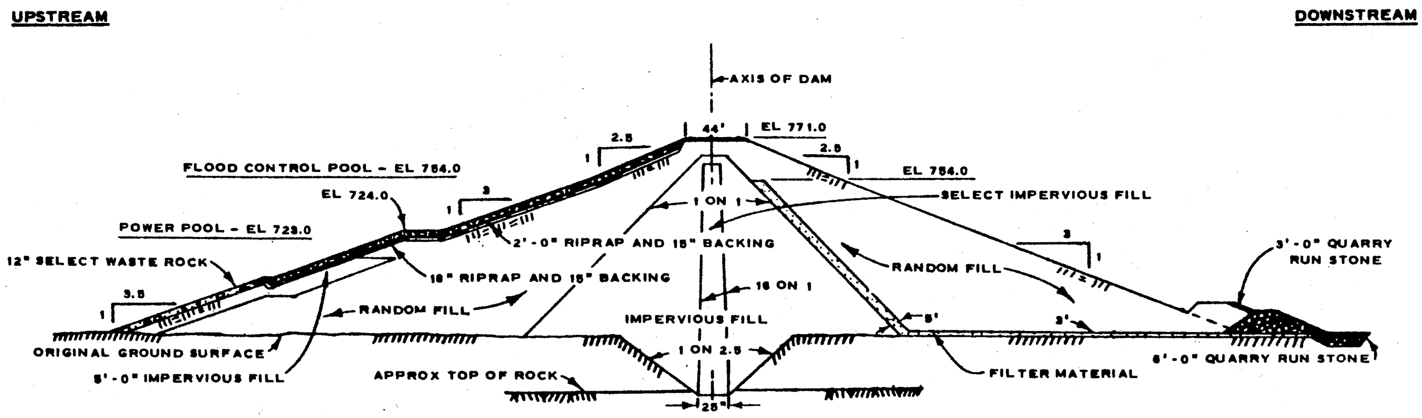


VALLEY CROSS SECTION

Source: (192)

Figure 70. Whitlow Ranch Dam, Queen Creek, Arizona.



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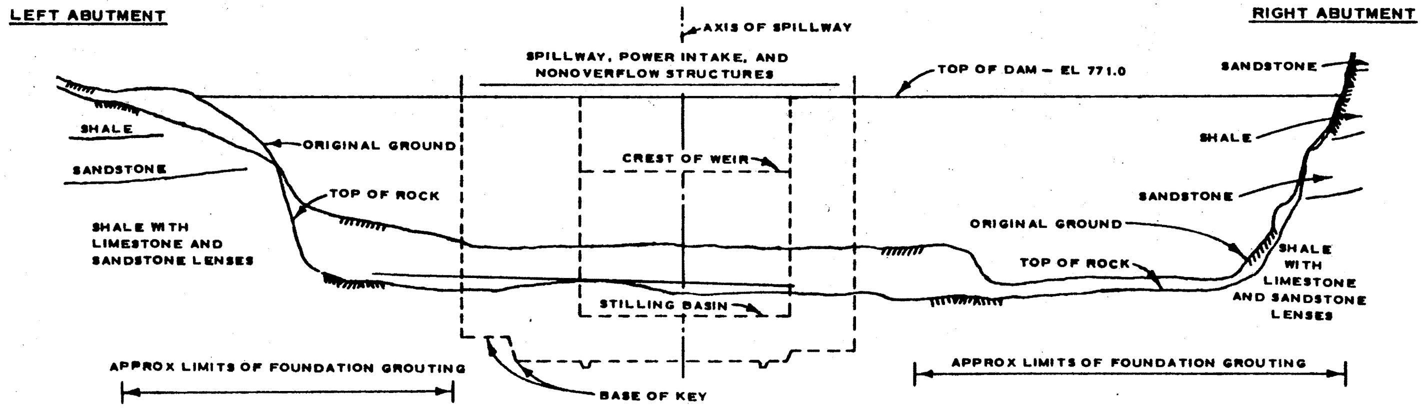
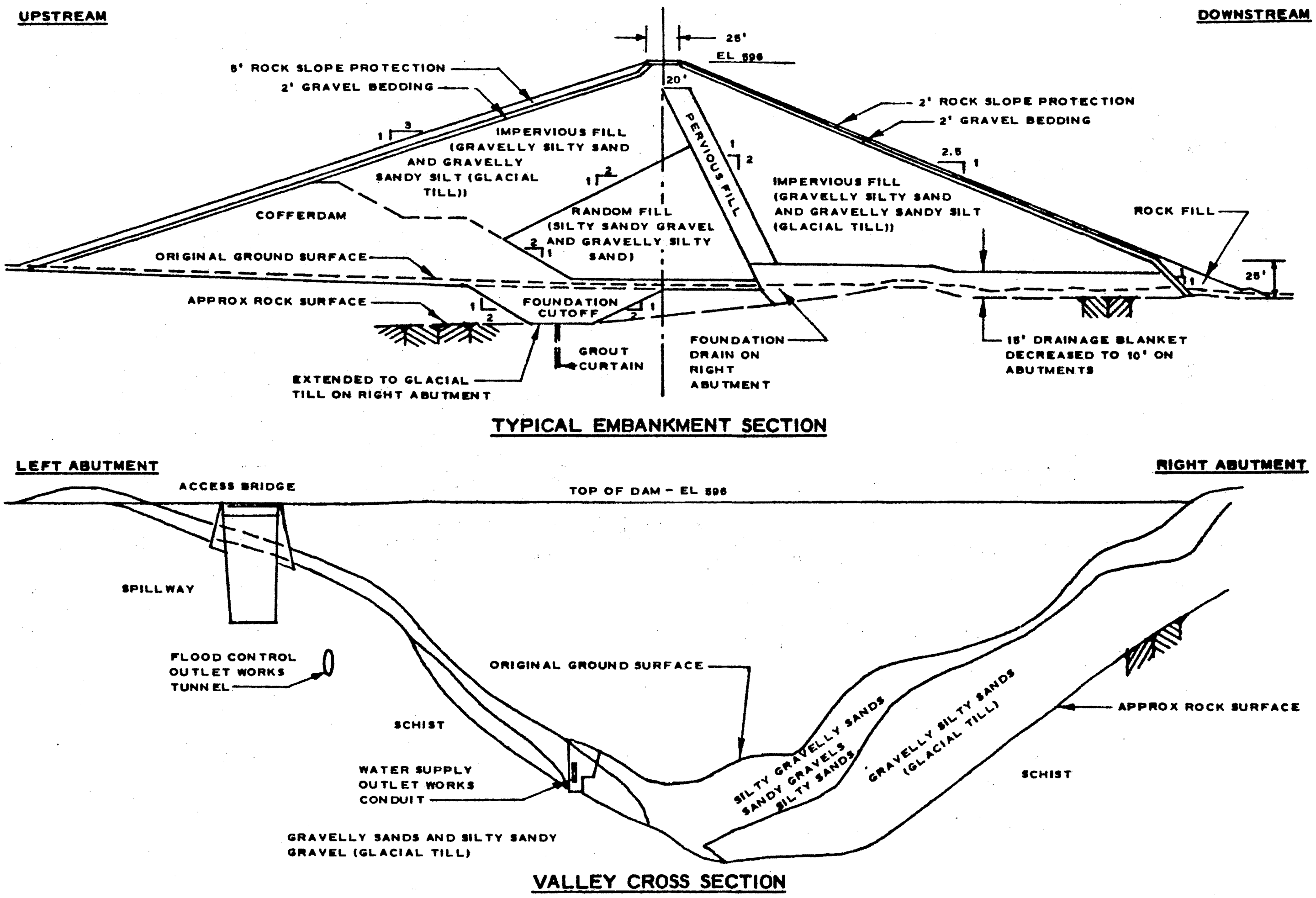


VALLEY CROSS SECTION

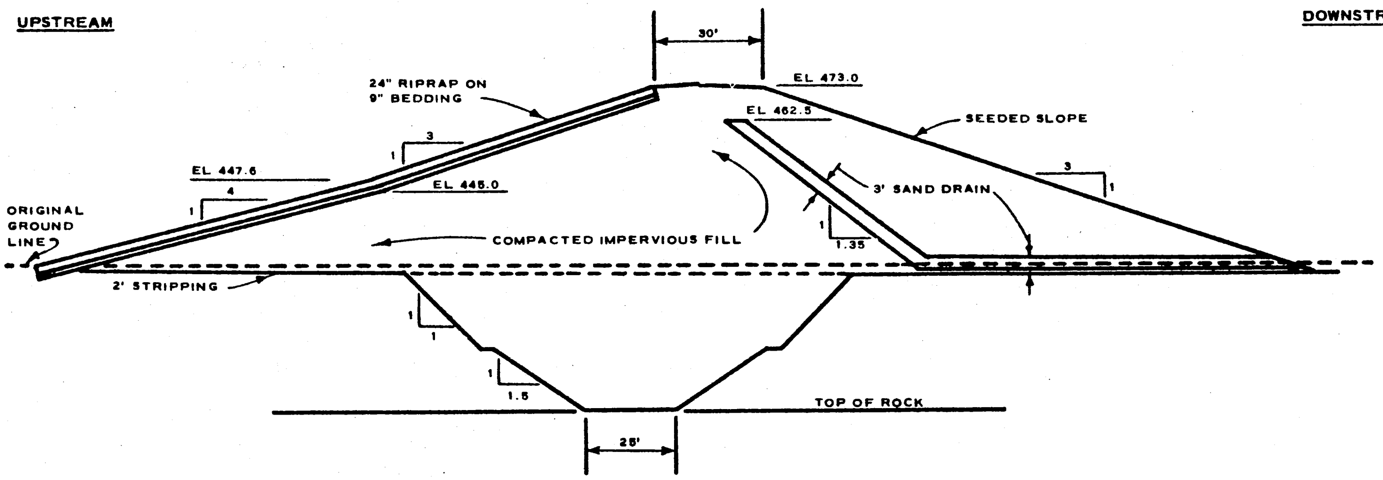
Source: (171)

Figure 71. Keystone Dam, Arkansas River, Oklahoma.



Source: (74)

Figure 72. Littleville Dam, Middle Branch, Westfield River, Massachusetts.



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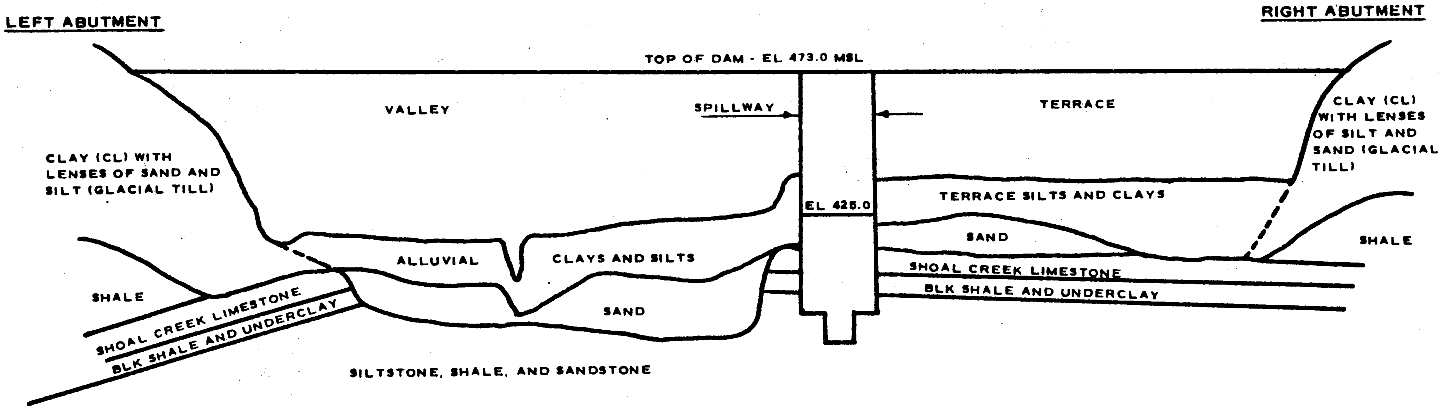
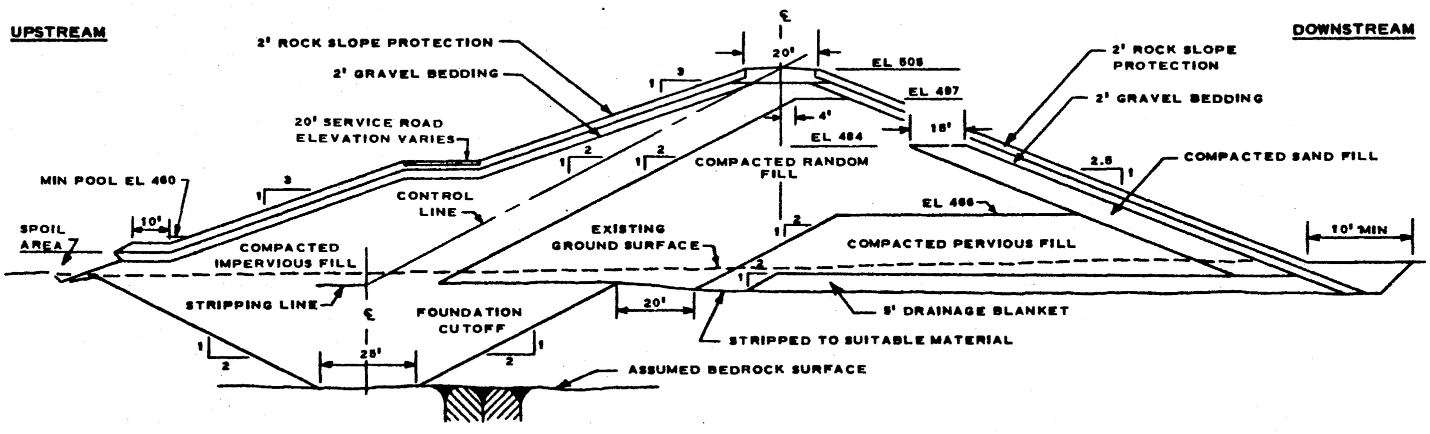


VALLEY CROSS SECTION

Source: (143)

Figure 73. Carlyle Dam, Kaskaskia River, Illinois.



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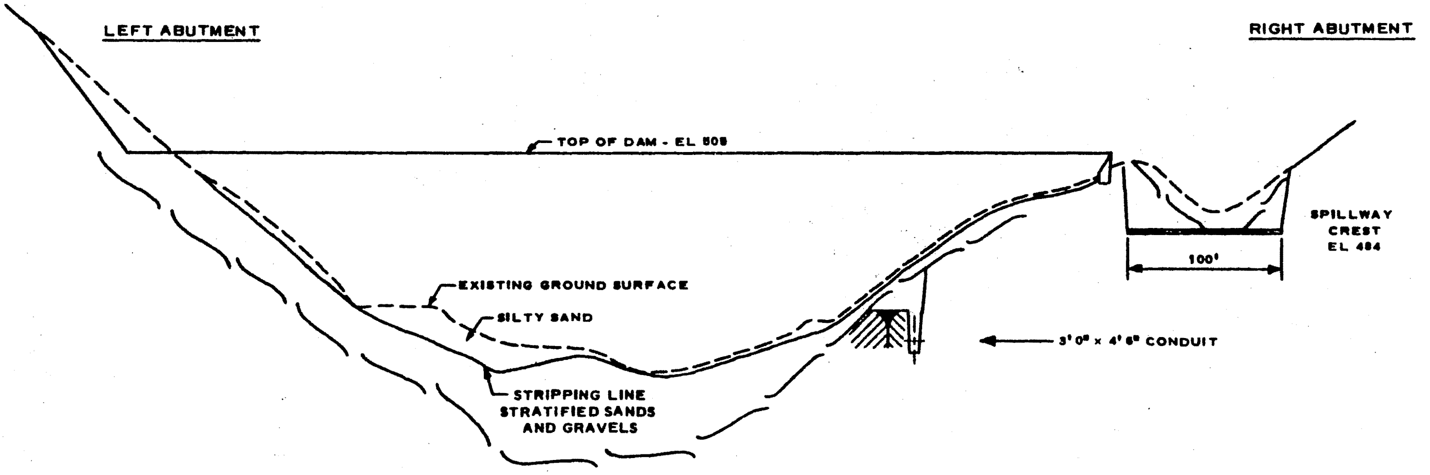
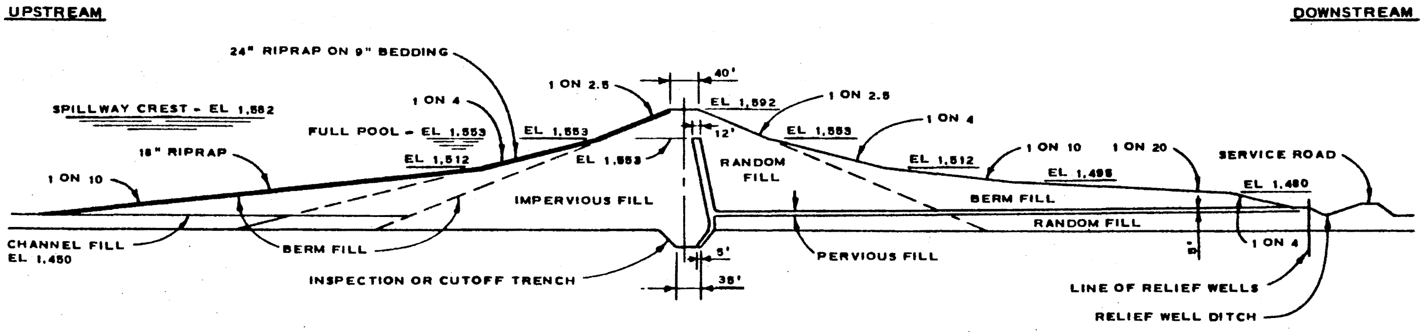


VALLEY CROSS SECTION

Source: (162)

Figure 74. Hancock Brook Dam, Hancock Brook, Connecticut.



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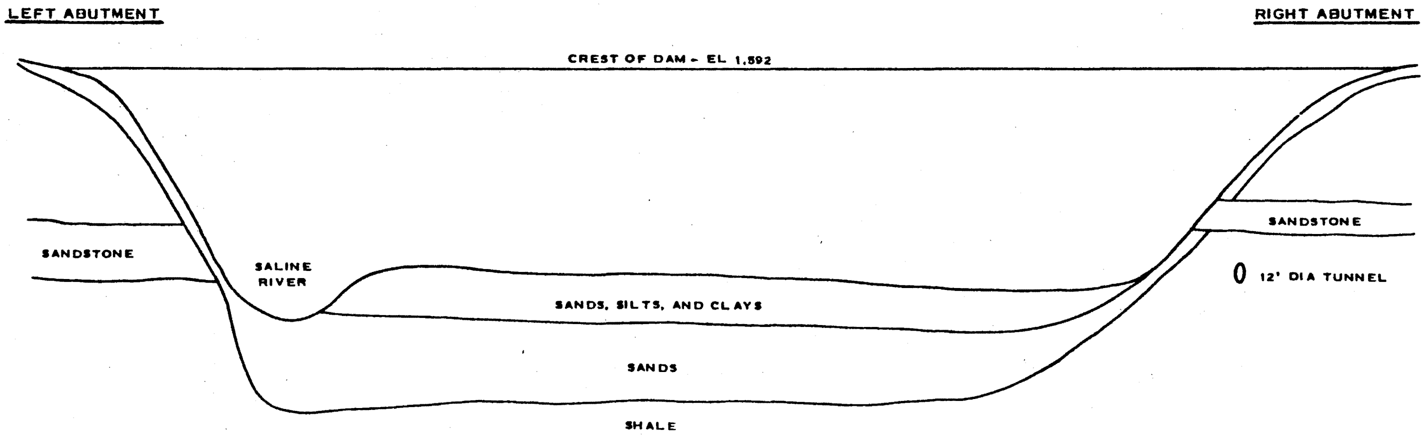
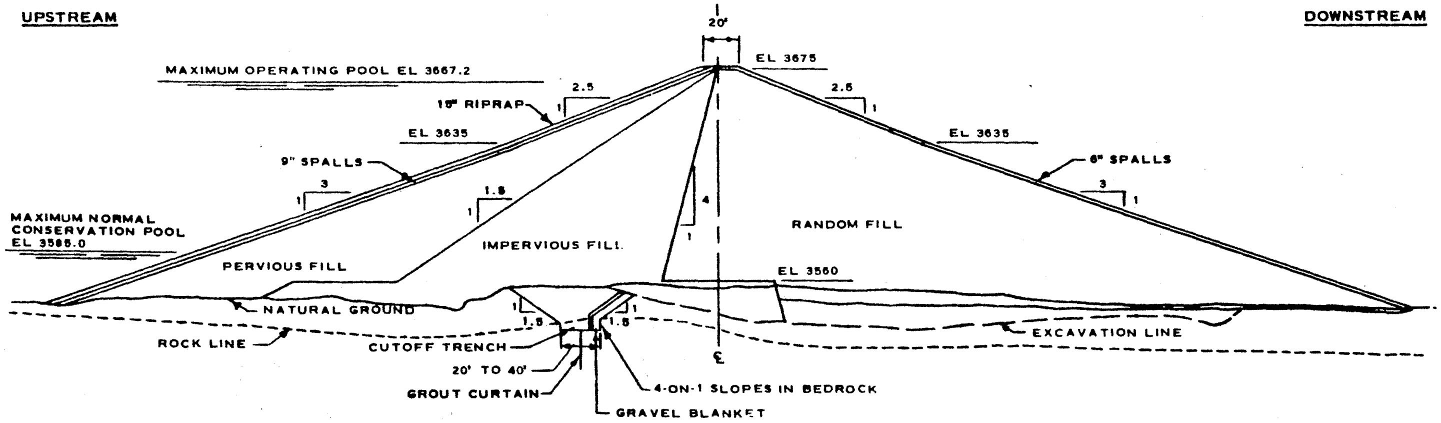


VALLEY CROSS SECTION

Source: (102)

Figure 75. Wilson Dam, Saline River, Kansas.



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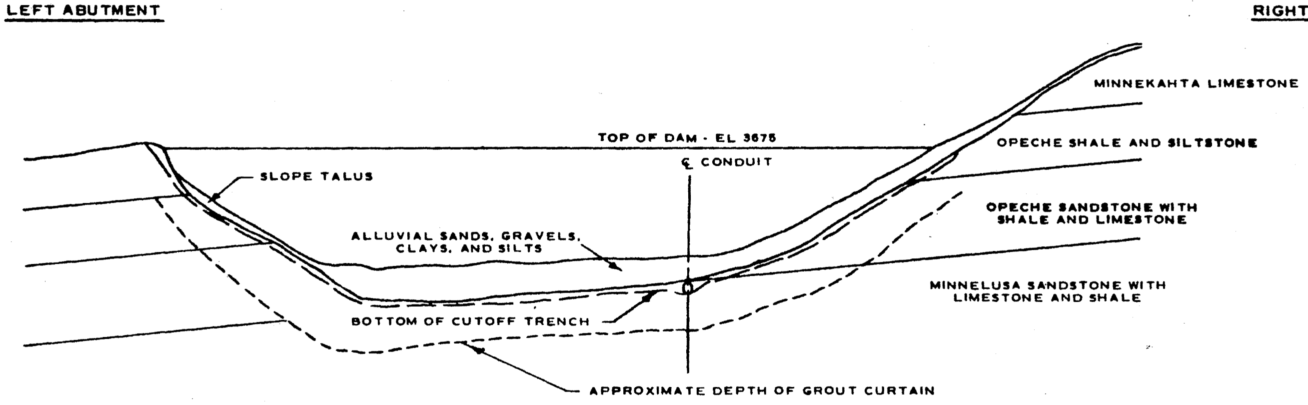
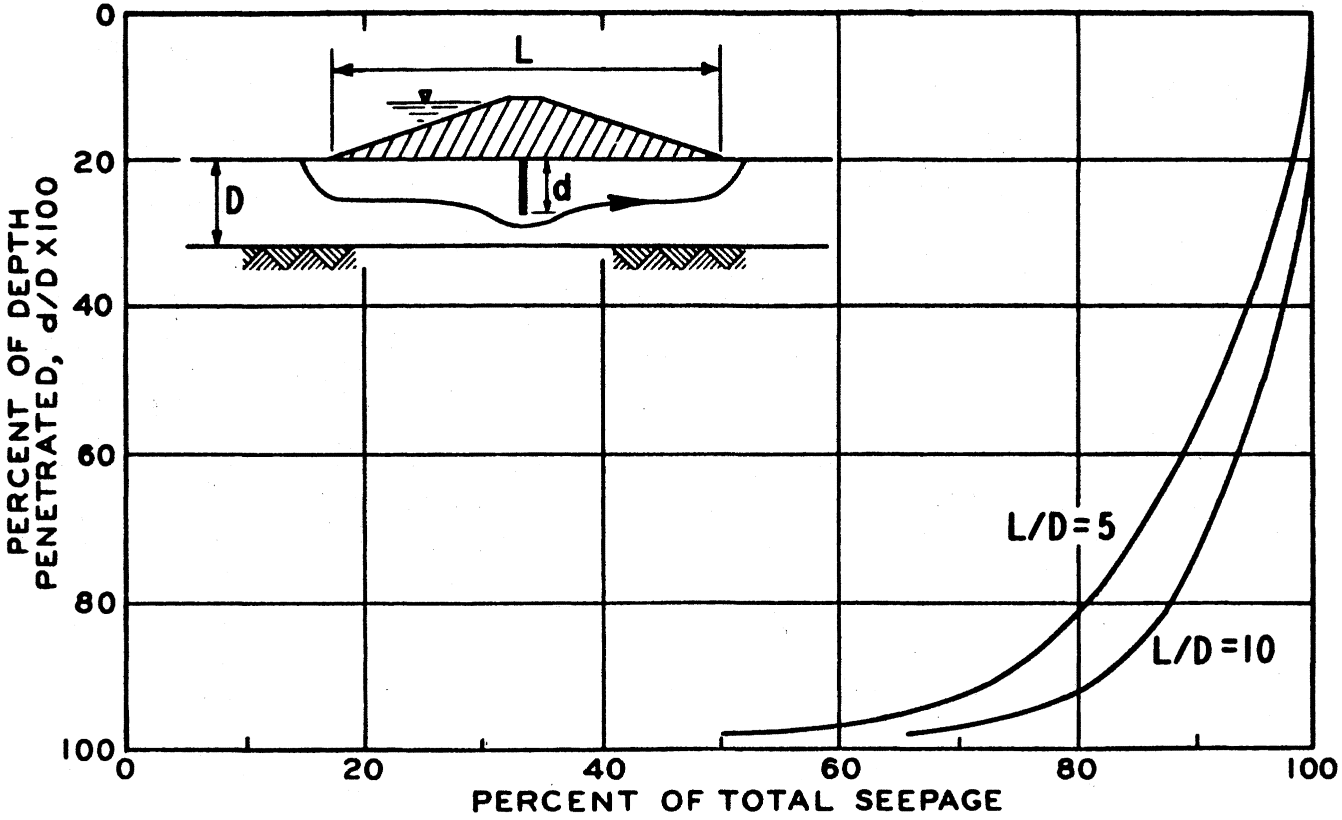


VALLEY CROSS SECTION

Source: (118)

Figure 76. Cold Brook Dam, Cold Brook Creek, South Dakota.

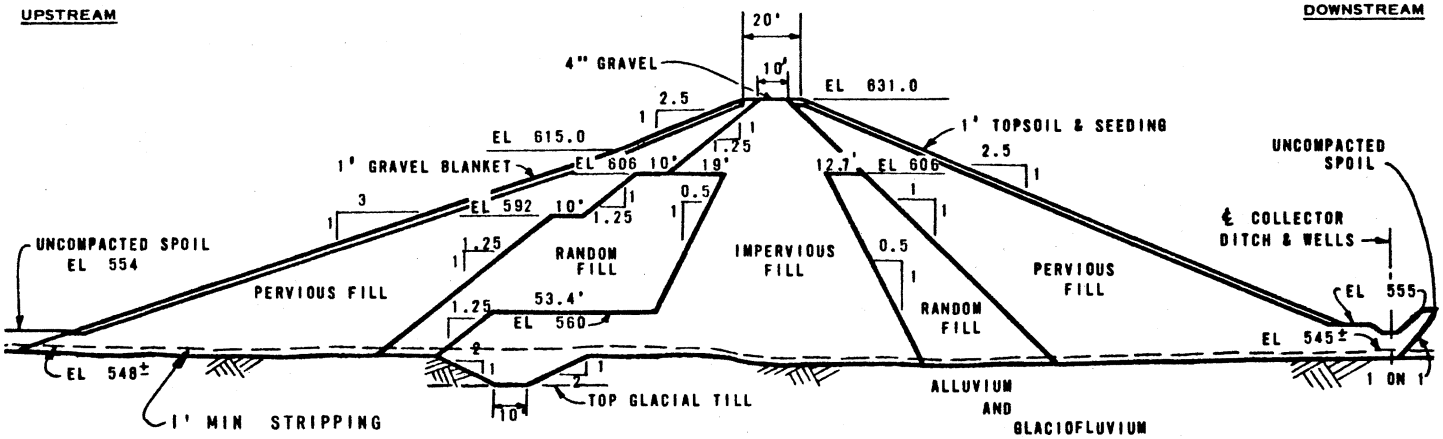


Source: (42)

Figure 77. Effectiveness of Partial Cutoffs in Reducing Underseepage.

Farmdale Dam in Illinois, Figure 78, and Cherry Creek Dam in Colorado, Figure 79. The foundation at Farmdale Dam consists of a shallow, near-surface stratum of highly erodable silts and fine sands overlying relatively impervious glacial till which in turn overlies a deep stratum of very pervious sands and gravels. A partial cutoff trench was excavated through the near-surface stratum to the top of the glacial till in order to reduce the potential of piping within the silts and fine sands. Control of underseepage and corresponding uplift pressures within the basal sands and gravels is through relief wells at the embankment toe (155). The foundation of Cherry Creek Dam consists of alluvial sand overlying a highly erratic rock surface. The maximum depth of the sand is approximately 110 feet within a narrow hidden valley. A cutoff trench was excavated to rock or to maximum depth of 50 feet which provided positive underseepage cutoff for all of the foundation except the hidden valley. Control of underseepage through the pervious sands remaining in the hidden valley is through relief wells (144).

Multiple cutoff trenches and multiple inspection trenches were constructed beneath early earth dams of the Bureau of Reclamation. While opposing the more modern practice of constructing a single cutoff or inspection trench, these multiple trenches provided more assurance of reduction of seepage through problem soils. The twin cutoff trenches beneath Sherbourne Lakes Dam and Palisades Dam provide positive cutoff of underseepage through shallow, pervious surface layers and are founded on strata of less pervious soils. The four inspection trenches at Rye Patch Dam and Bull Creek Dam only provide cutoff of localized leakage which might otherwise occur through the



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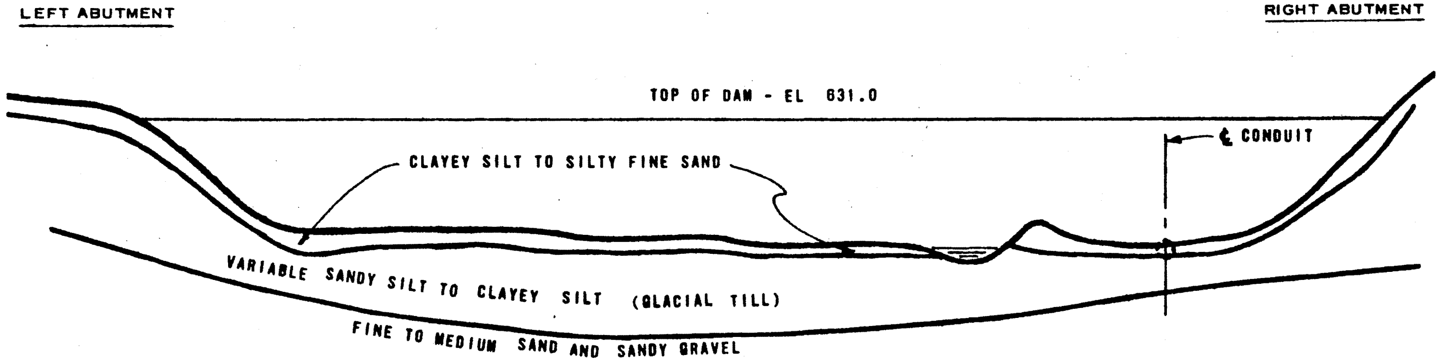
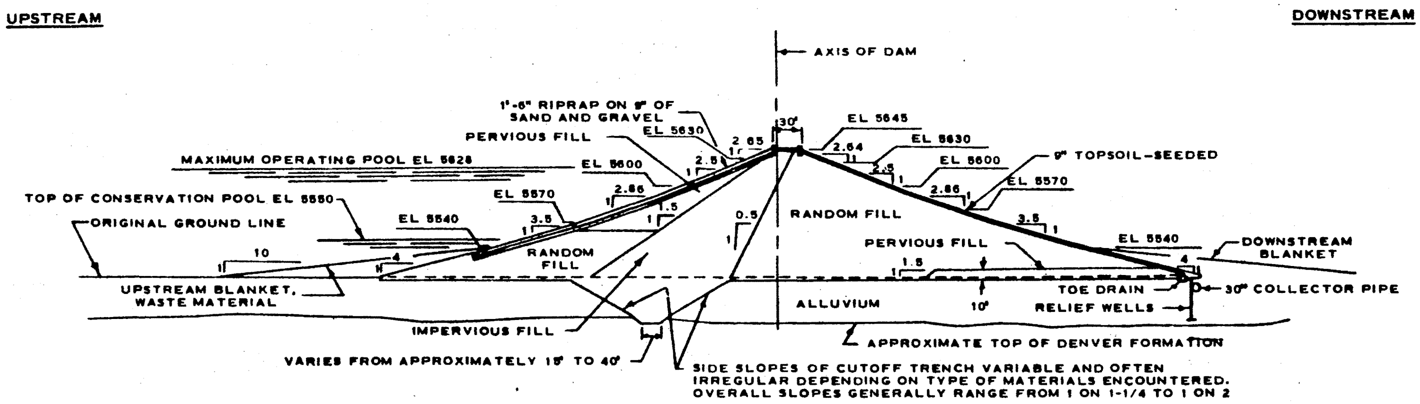


VALLEY CROSS SECTION

Source: (155)

Figure 78. Farmdale Dam, Farm Creek, Illinois.



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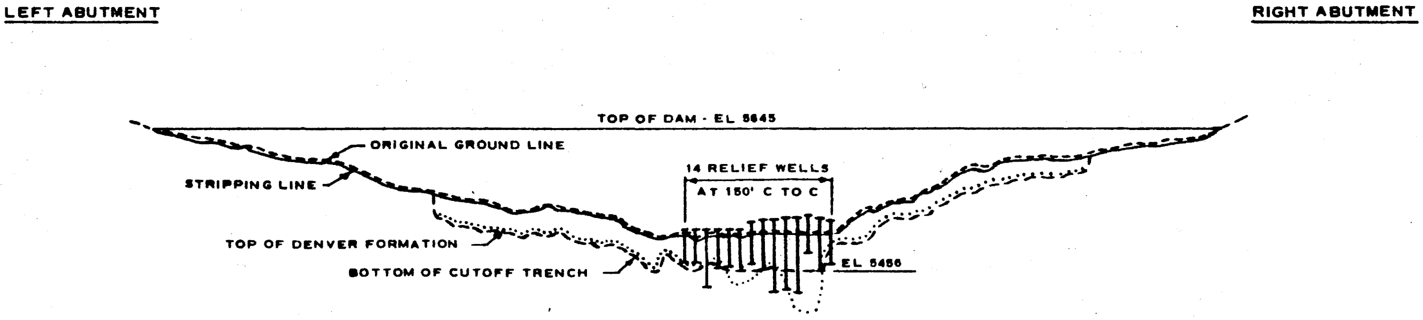


VALLEY CROSS SECTION

Source: (144)

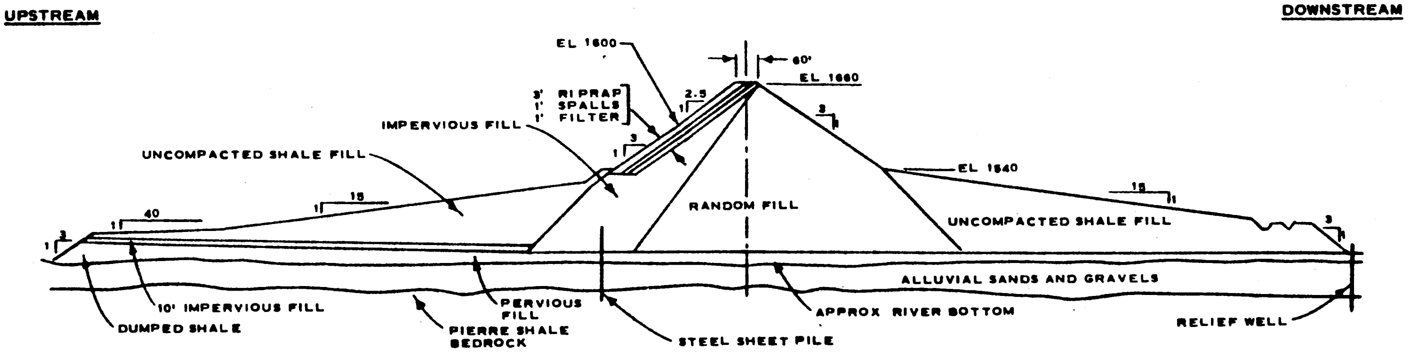
Figure 79. Cherry Creek Dam, Cherry Creek, Colorado.

surface soil layer (211).

The minimum treatment, in addition to obligatory foundation stripping, should be the excavation of an inspection trench located at or near the dam axis. The inspection trench should be from 5 to 10 feet deep, wide enough to accommodate excavating and hauling equipment, and extend through the entire alignment of the embankment. An inspection trench provides a limited visual examination of the foundation soils for additional evaluation which could result in a deeper excavation. Compacted impervious backfill in the inspection trench provides a seepage barrier through the seasonally active surface layer and prevents localized underseepage through dessication cracks, root cavities and animal burrows (57).

Steel Sheet Piling

Steel sheet piling has lost much of its appeal as a positive cutoff. Although always relatively expensive and therefore only a limited contender, experience has shown that the efficiency of a sheet pile cutoff is quite low because of seepage through the interlocks between individual piling. Under the best conditions, including compounds to seal the interlocks, the efficiency may be expected to range from 80 to 90 percent (210). The measured efficiency of sheet pile cutoffs beneath several of the large Corps of Engineers dams on the Missouri River was determined to be as low as 10 percent (38). Because of low efficiency, sheet pile cutoffs are almost always used in conjunction with major seepage control measures such as foundation drains or relief wells. The embankment section of Oahe Dam in South Dakota as shown in Figure 80 provides a typical illustration of the use



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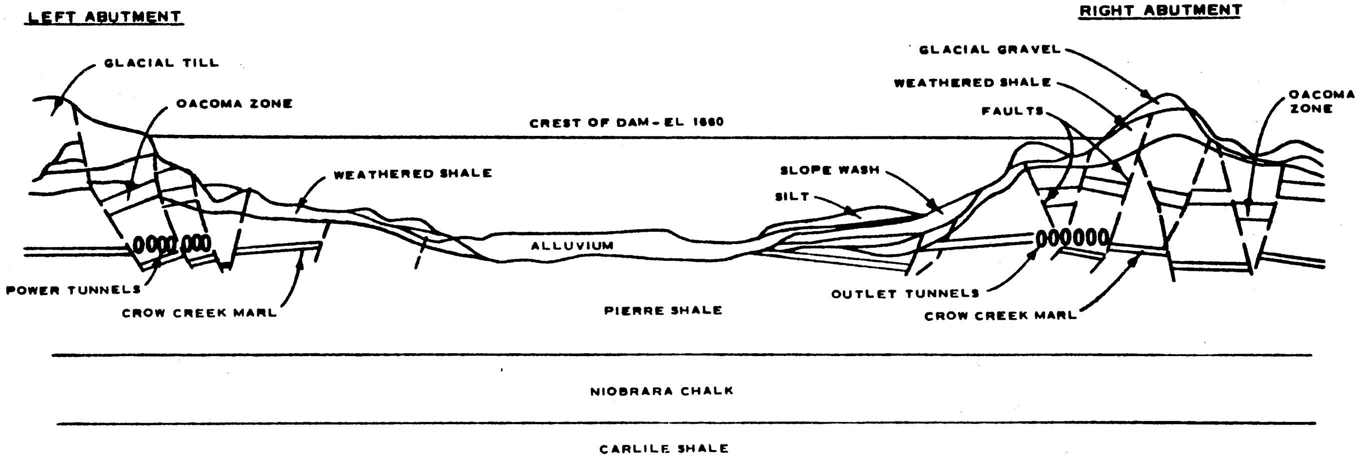


VALLEY CROSS SECTION

Source: (105)

Figure 80. Oahe Dam, Missouri River, South Dakota.

of steel sheet piling.

The efficiency of any cutoff can be expressed by the ratio,

$$E = \frac{h'}{h} \tag{4.1}$$

where h' is equal to the difference between piezometric elevations immediately upstream and downstream of the cutoff, and where h is the total head equal to the difference between reservoir level and tailwater level (62). Efficiency calculations at selected locations along the sheet pile cutoff at Denison Dam in Texas after completion of the project indicated efficiencies ranging from 5 to 62 percent (66). Table IX presents efficiency calculations for the sheet pile cutoff at Denison Dam. The efficiency of the sheet pile cutoff at Station 138+00 had doubled during the 19-year interval between piezometric readings. The increase in the efficiency of sheet pile cutoffs with time is due primarily to corrosion of the interlocks, and a similar increase in efficiency was also recorded at Fort Peck Dam and Garrison Dam (38).

Applications of steel sheet piling for underseepage cutoffs are limited to foundations comprised of silt, sand and fine gravel because of driving problems in coarser materials. Additionally, the impervious stratum to which the sheet piles are to be driven should be penetrable by the sheet pile for at least a few inches. The contract should not be so erratic or sloping that the individual sheet piles cannot penetrate for their full width. The sheet piling should extend a minimum of five feet above the stripped foundation for proper interfacing with the impervious core of the embankment (210). Figure 81 shows the driving of sheet piles to form a cutoff at Denison Dam at approximately Station 138+00.

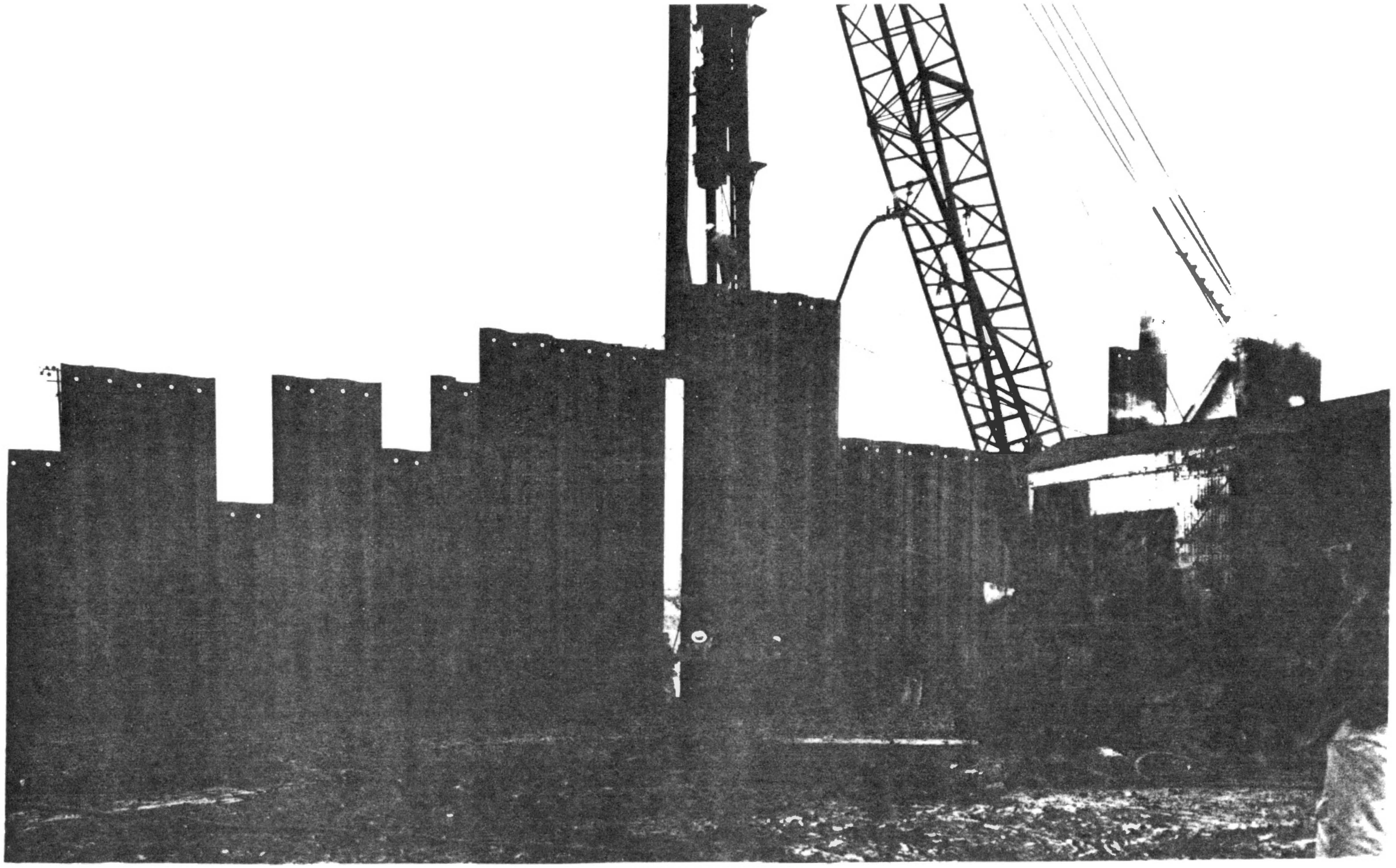
TABLE IX

EFFICIENCY CALCULATIONS FOR THE SHEET PILE CUTOFF AT DENISON DAM, TEXAS

Item	Description		Tabulation	
1.	Station		138+00	138+00
2.	Date		Mar 47	May 66
3.	Reservoir elevation		612.6	617.6
4.	Piezometer numbers (see note below)	a.	30	30
		b.	31	31
		c.	33	33
5.	Elevation of water in piezometers	a.	605.5	608.3
		b.	593.9	590.6
		c.	582.6	581.8
6.	Distance between piezometers a and b, feet		120	120
7.	Distance between piezometers b and c, feet		200	200
8.	Difference in elevation of water in piezometers b and c (considered to be head loss in foundation between these piezometers)		11.3	8.8
9.	Average foundation head loss gradient (Item 8 divided by Item 7)		0.055	0.044
10.	Foundation head loss between piezometers a and b (Computed using gradient in Item 9)		6.6	5.3
11.	Difference in elevation, reservoir and water in c (Considered to be total net head on dam)		30.0	35.8
12.	Difference in elevation of water in piezometers a and b (Considered to be total head loss through sheet piling, including head loss in foundation)		11.6	17.7
13.	Head loss through sheet piling, corrected for head loss in foundation (Item 12 less Item 10)		5.0	12.4
14.	Head loss through sheet piling, expressed as a percent of total net head on dam (Item 13 divided by Item 11)		17	35

Note: Piezometers a, b, and c are those nearest the sheet piling on the upstream side, nearest the sheet piling on the downstream side, and farthest downstream, respectively.

Source: (66)(149)



Source: (149)

Figure 81. Driving of Steel Sheet Piling as Cutoff at Denison Dam, Texas.

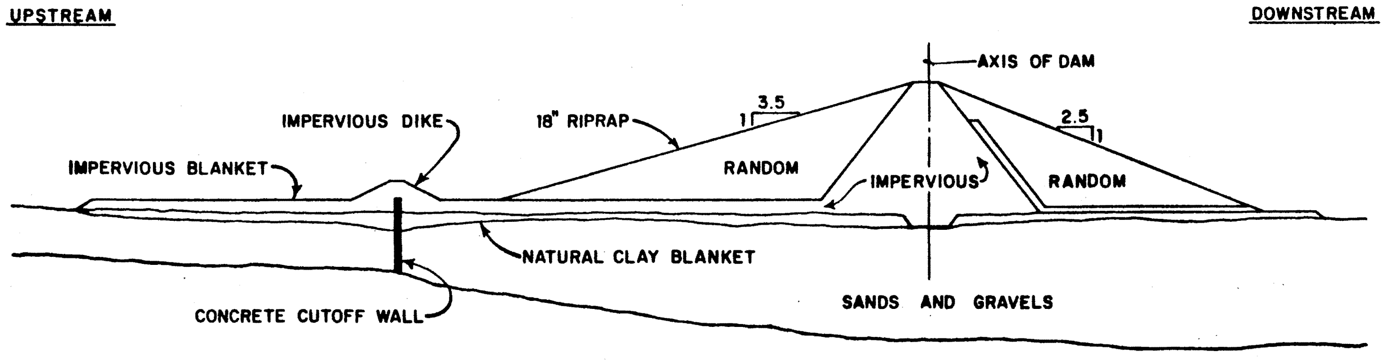
Steel sheet piling is recommended primarily because of its high strength which is required during driving. For this reason steel sheet piling has been used exclusively beneath earth dams. However, wooden sheet piling was used through the valley section of Imperial Dam and steel sheet piling was used only in the abutments. Permanently submerged, the wooden sheet piling should be expected to resist decomposition and provide satisfactory service. Imperial Dam is a slab and buttress dam, completed by the Bureau of Reclamation in 1940 for water conveyance diversion into the All-American Canal to California (211).

Concrete Walls

Although somewhat common in Europe, concrete walls as positive cutoffs in foundation soils have had very limited use in the United States. A concrete wall creates a brittle, rigid heterogeneity within the foundation soil and may be subjected to cracking during consolidation of the soil. Consequently, the location of a concrete cutoff wall should be upstream of the embankment. Locating the concrete wall upstream of the embankment will not necessarily provide easy access for remedial repairs because of innundation even by the minimum-drawdown reservoir pool.

Kinzua Dam at Pennsylvania, formerly designated as Allegheny Reservoir Dam and completed in 1965, is the only earth dam within the United States constructed in recent years with a concrete cutoff wall to prevent underseepage through foundation soils. The concrete cutoff wall at Kinzua Dam is immediately upstream of the embankment and approximately 1,130 feet long with a minimum thickness of 2.5 feet and a maximum depth of 186 feet. The wall extends downward through the overburden to a minimum of two feet into underlying bedrock consisting of shale and siltstone. The wall is connected to the impervious core of the embankment by an upstream impervious blanket (106). A typical embankment section of Kinzua Dam is shown in Figure 82. The very pervious nature of the foundation soils at Kinzua Dam and the depth to bedrock were the significant factors in the selection of a concrete cutoff wall. The concrete wall was constructed as cast-in-place panels within a trench excavation stabilized with bentonite slurry, after the embankment was completed.

The Bureau of Reclamation has utilized three distinct types of concrete walls to provide reduction of underseepage beneath earth dams. The most ambitious type, at two dams, involved concrete corewalls founded 10 feet into rock and extending through the pervious foundation and through the full height of the embankment. The first was at Tieton Dam in Washington, completed in 1925, and within a 235-foot hydraulic fill embankment. The inward flow of the hydraulic-fill placement created a clay puddle core adjacent to the concrete corewall. The second was at American Falls Dam in Idaho, completed in 1927, and within a 94-foot rolled-earthfill embankment (211). Concrete corewalls were not used again in early dams by the Bureau of Reclamation because of construction problems related to the dewatering and shoring of the open excavations in soils (215). Developments in bentonite slurry and tremie-placement of concrete have eliminated the construction problems for concrete corewalls. However, modern analyses of deformations resulting from settlement and consolidation of earth embankments have identified compatibility problems between rigid concrete walls and compressible earthfill embankments.



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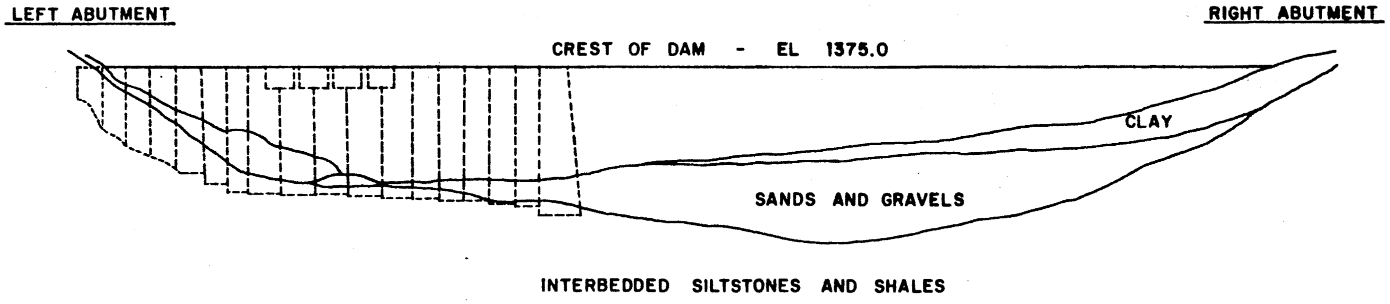


VALLEY CROSS SECTION

Source: (106)

Figure 82. Kinzua Dam, Allegheny River, Pennsylvania.

A second and less ambitious type of concrete wall involved the construction of a concrete cutoff wall founded in rock and extending only through the pervious soil foundation. The concrete cutoff wall at Moon Lake Dam in Utah, completed in 1938, is located beneath the upstream impervious zone of the embankment, while the concrete cutoff wall at McKay Dam in Oregon, completed in 1927, is located along the upstream toe of the embankment and is a forerunner of the cutoff wall at Kinzua Dam (211).

The third distinct type of cutoff wall used by the Bureau of Reclamation was a short wall, from 5 to 15 feet in height and embedded 3 to 5 feet in rock, to provide cutoff of localized leakage which might occur at the contact between compacted earthfill and underlying foundation rock. As many as four parallel concrete cutoff walls have been constructed on rock foundations which had been entirely stripped of overburden. For foundations which were not stripped of overburden, one or two walls were located along the bottoms of cutoff trenches. Almost all Bureau of Reclamation earth dams constructed before the late 1950's and where foundation rock was exposed, either through overburden stripping or cutoff trench excavation, were provided with these concrete cutoff walls. Presently, both the Bureau of Reclamation and the Corps of Engineers prohibit this type of concrete wall because of problems related to equipment compaction of earthfill near the wall (128)(210).

A continuous row of overlapping concrete piers drilled several feet into rock can produce a satisfactory concrete cutoff wall. One of the deepest cutoffs of this type is located beneath the upstream cofferdam of the Manicouagon V Project in Quebec. The cutoff extends through alluvial sediments to a maximum depth of 250 feet and has been shown through piezometric observations to be practically impervious (30).

Slurry Trench Cutoffs

The construction of slurry trench cutoffs is a relatively new method to prevent underseepage through pervious foundation soils. The first major earth dam constructed wtih a slurry trench cutoff was Wanapun Dam in Washington State which was completed in 1962. Subsequent applications are presented in Table X.

Slurry trench cutoffs are well suited to either homogeneous or highly stratified pervious soils and have been demonstrated to provide a very effective cutoff. However, slurry trench cutoffs are unsuited to foundation soil overlying open-jointed rock or containing boulders or talus blocks. The initial slurry cannot stabilize trench sides which contain large voids into which the slurry can readily navigate. Caving of the lower portion of the trench is possible which could leave open seepage paths through the cutoff (128).

Excavation of the vertical-sided trench may be accomplished with draglines, backhoes, clamshells, or ditching machines. Once the trench has been excavated a sufficient distance to allow backfill operations to begin, progressive excavation and backfilling may commence to provide a continuous slurry trench cutoff for the length of the dam (126). The sequence of construction of a slurry trench cutoff is illustrated in Figure 83.

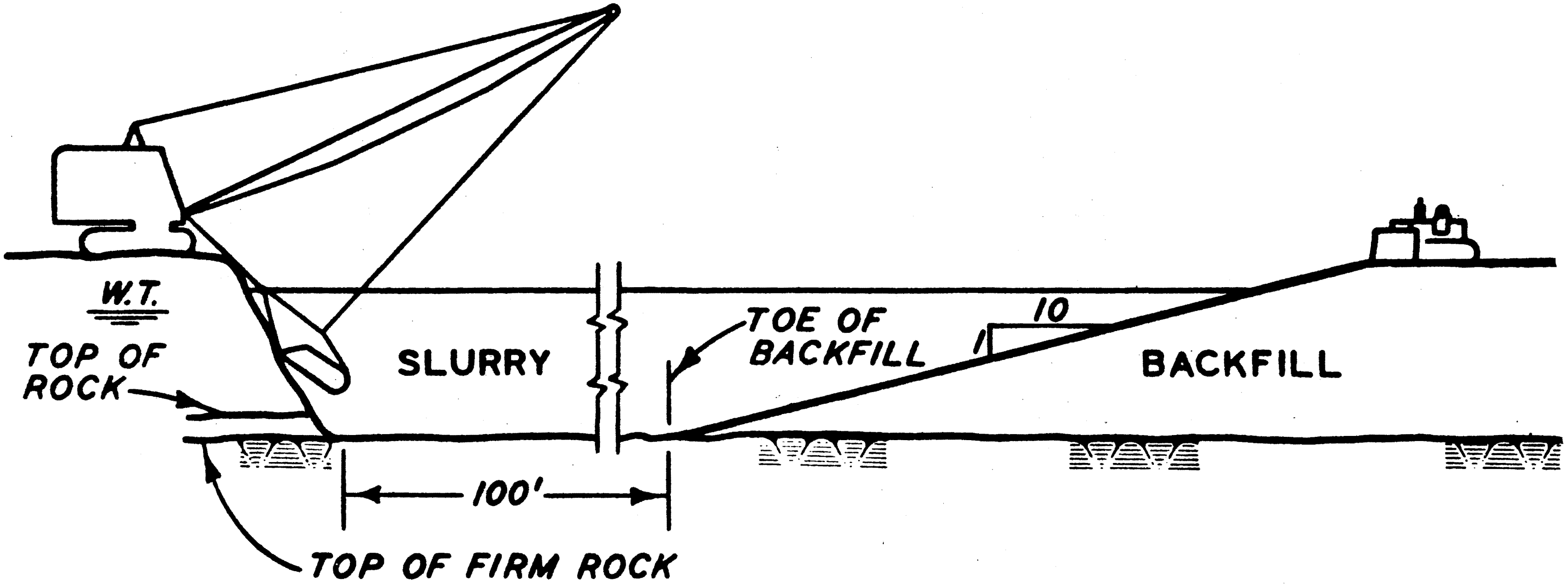
The location and dimensions of the slurry trench cutoff depends on the specific requirements of a particular site and embankment design. The two major considerations for selecting an appropriate location of

TABLE X

EARTH DAMS WITH SLURRY TRENCH CUTOFFS

Project	Foundation Material	Trench Width	Maximum Head	Remarks
Wanapum Dam, Columbia River, Washington, U.S. Owner: Public Utility District No. 2 of Grant County	Sandy gravels and gravelly sands underlain by open-work gravels.	10 ft. Central core.	88.5 ft.	Constructed in 1959-62. Maximum depth of cut-off 190 ft.
Duncan Lake Dam, Duncan River, British Columbia, Canada. Owner: British Columbia Hydro and Power Authority.	Surface zone of sands and gravels over silt to fine silty sand with some silty clay.	10 ft. Upstream berm.	102 ft.	Constructed in 1965-66. Maximum depth 60 ft.
West Point Dam, Chattahoochee River, Georgia and Alabama, U.S. Owner: Corps of Engineers.	Upper stratum of alluvial soil: alternating layers of clay, silt, sand, and gravel. Lower stratum of residual soil: silty sand.	5 ft. Upstream blanket	61 ft.	Constructed in 1966. Maximum depth 60 ft.
Saylorville Dam, Des Moines River, Iowa, U.S. Owner: Corps of Engineers.	Surface zone of impervious sandy clay over pervious zone of sand and gravelly sand.	8 ft. Upstream berm.	93 ft.	Constructed in 1969. Maximum depth 60 ft.
Wells Dam, Columbia River, Washington, U.S. Owner: Public Utility District No. 1 of Douglas County.	Pervious gravels.	8 ft. Central core.	70 ft.	Constructed in 1964. Maximum depth 80 ft.
Yards Creek Lower Reservoir, New Jersey, U.S. Owner: Public Service Electric and Gas Company, Jersey Central Power and Light, New Jersey Power and Light Co.	Sands, gravels, cobbles, and boulders.	8 ft Central core.	55 ft	Constructed in 1964. Maximum depth 40 ft.
Camanche Dam-Dike 2, Mokelumne River, California, U.S. Owner: East Bay Municipal Utility District.	Upper stratum: clayey silts, silts, and clayey sands. Lower stratum: fine sand over well-graded gravel.	8 ft.	135 ft.	Constructed in 1966. Maximum depth 95 ft. Maximum head depends on tailwater elevation.
Addicks Dam, Buffalo Bayou, Texas, U.S. Owner: Corps of Engineers.	Sandy clays and silty sands in embankment and silty sands in foundation.	3 ft Upstream slope.	65 ft.	Under construction to provide seepage control within existing earth dam.

Source: Adapted from (210)



Source: (126)

Figure 83. Progressive Excavation and Backfilling Operations for Slurry Trench Cutoff Construction.

the slurry trench cutoff are the compressibility and strength of the slurry trench backfill. Locating the slurry trench cutoff beneath a central embankment core provides the maximum settlement of the embankment as a compensation for consolidation of the slurry trench backfill. However, the stability of the embankment may be drastically reduced. The Corps of Engineers recommends locating the slurry trench cutoff upstream of the embankment and connected to the impervious core by an upstream impervious blanket (128). This recommendation was incorporated into the design and construction of West Point Dam in Georgia and Saylorville Dam in Iowa. The location of slurry trench cutoffs at Aliceville Lock and Dam and Columbus Lock and Dam, features of the Tennessee-Tombigbee Waterway, are beneath the central impervious cores; however, neither of these two dams is expected to experience major differential hydraulic heads (1).

The width of the slurry trench cutoff should be sufficient to prevent a blowout type of failure during which the slurry trench backfill is forced into the pervious foundation soil under the maximum potential hydraulic head. A conservative factor of safety, greater than for conventional design, is appropriate to reflect construction imperfections (223). A general rule-of-thumb is to provide one foot of width for each 10 feet of differential hydraulic head (1). Slurry trench cutoffs greater in width than eight feet exhibit a more pronounced settlement (223).

The backfill material should contain enough fines to produce a relatively impervious membrane and enough coarse particles to reduce settlement to a minimum and to provide some degree of shear strength (128). A minimum of from 10 to 20 percent fines, preferably clay, in the slurry trench backfill should be adequate to produce a coefficient of permeability within the slurry trench cutoff of almost 10^{-6} cm/sec (1).

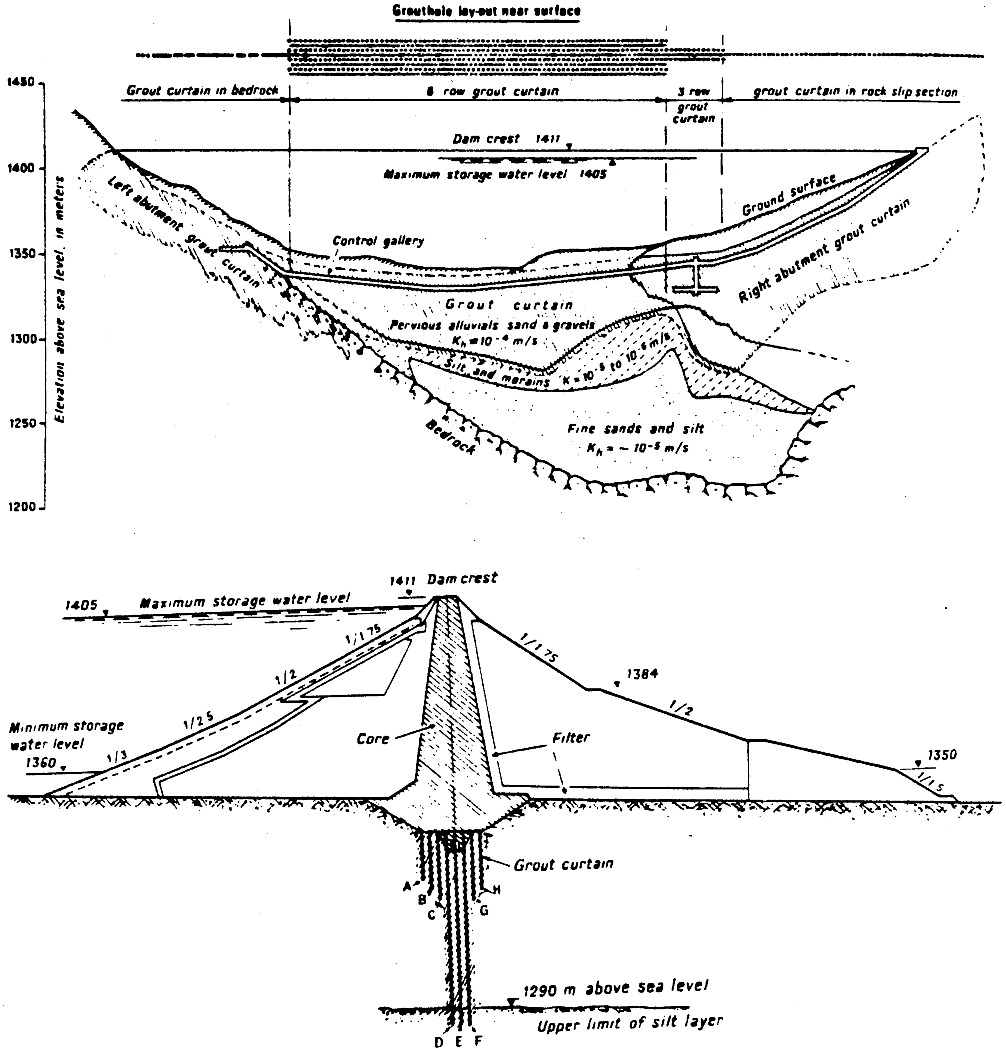
Alluvial Grouting

Early attempts at grouting pervious foundation soils used cement grout and failed because of thickening of the grout. The lean cement grouts lost water when in contact with sand and fine gravels. Increasing the grouting pressure only served to hydraulically fracture the foundation. Chemical grouts can retain their initial viscosity but are prohibitive for large scale grouting because of the expense.

Grouts comprised of clay with admixtures have been developed by French engineers and adapted for use to provide underseepage cutoffs. Successful cutoffs have been achieved by alluvial grouting beneath Serre-Poncon Dam in France, Mattmark Dam in Switzerland, Durlassboden Dam in Austria, Aswan High Dam in Egypt, Karl Terzaghi Dam (formerly Mission Dam) in Canada, and Thulaga Dam in Scotland. Figure 84 shows the alluvial grouting beneath Durlassbaden Dam in Austria.

The only application of alluvial grouting at a dam within the United States was the grouting of terrace gravels at Rocky Reach Hydroelectric Power Project in Washington. Rocky Reach Project is located on the Columbia River and consists of a power house on the right bank and a concrete spillway adjoining the power house and extending across the river to the left bank. Alluvial grouting was conducted within the pervious gravels of the left abutment to reduce seepage around the end of the dam (60).

The important properties of an alluvial grout are viscosity,



Source: (12)

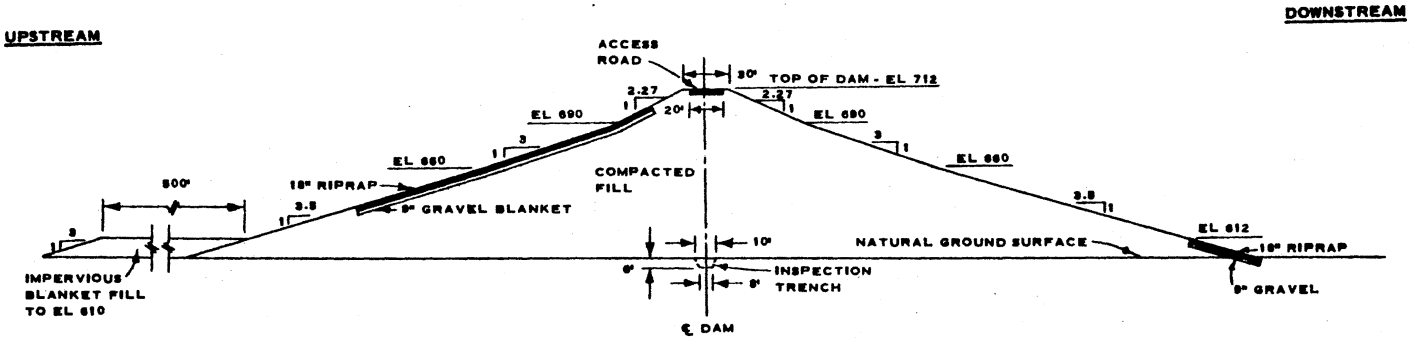
Figure 84. Durlassboden Dam, Austria.

rigidity and granular state. As the soil is found to become finer grained the grout must be more liquid: Binhamian suspensions in coarse soils; colloidal solutions in soils of medium grain size; and pure solutions in fine soils (12). Large scale tests have demonstrated that the coefficient of permeability of grouted alluvium is in the range 10^{-5} cm/sec, regardless of the coefficient of permeability prior to grouting (62).

Upstream Impervious Blankets

An upstream impervious blanket increases the flow path of underseepage through pervious foundation soils and thereby decreases the hydraulic gradient. A decrease in the hydraulic gradient will result in a reduction of underseepage quantities and a reduction of uplift pressures within the foundation. The upstream impervious blanket may tie directly into an upstream impervious zone of an embankment or tie into an impervious core through an impervious foundation blanket in the upstream portion of the embankment. An upstream impervious blanket is, in effect, an extension of a foundation blanket into the reservoir area. An upstream impervious blanket extending from a homogeneous embankment is illustrated by the typical section of Mansfield Dam in Indiana shown in Figure 85.

The thickness and length of an upstream blanket to significantly reduce underseepage is dependent upon the permeability of the blanket material, the stratification, permeability and depth of the pervious foundation, and the reservoir head. The effectiveness of a specific upstream impervious blanket may be analyzed by flow nets, electric analog methods, or by finite element analyses. However, for design



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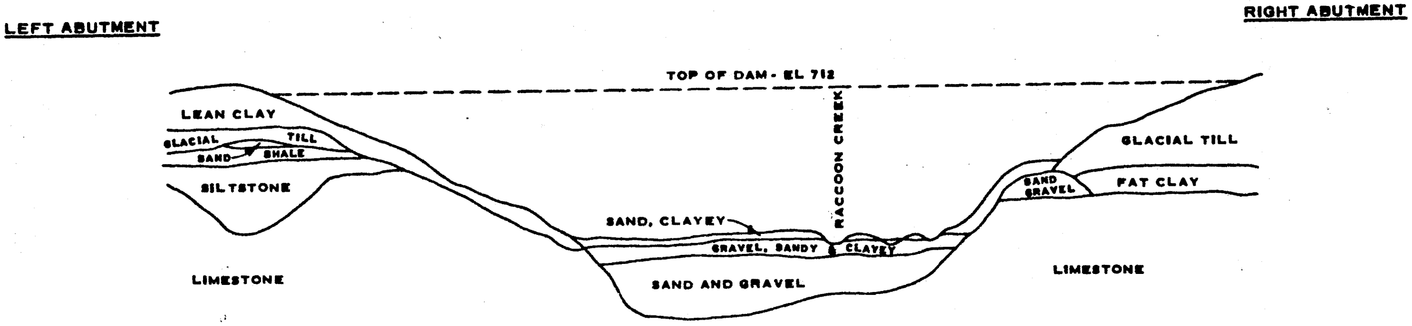


VALLEY CROSS SECTION

Source: (173)

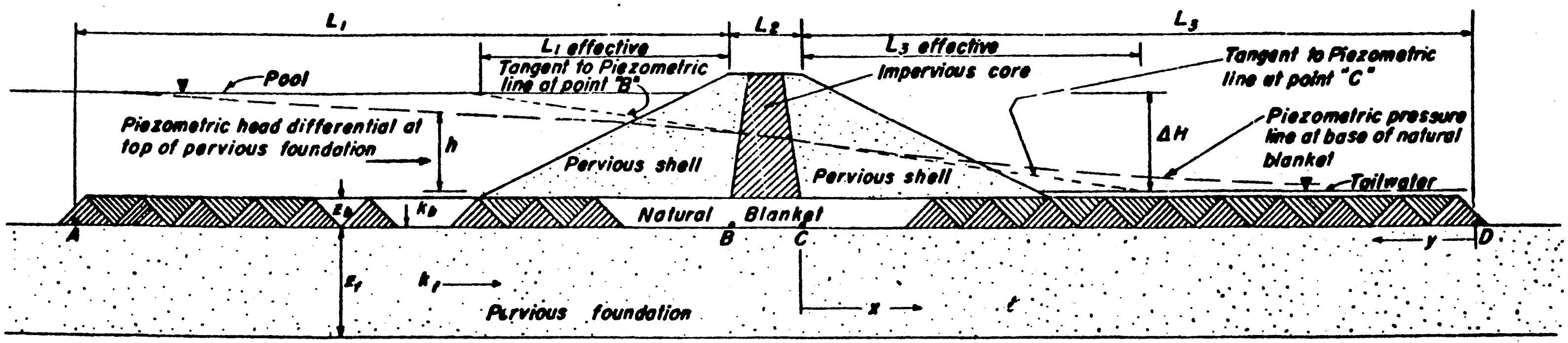
Figure 85. Mansfield Dam, Raccoon Creek, Wabash River, Indiana.

purposes, all of these methods are cumbersome, tedious, and very time-consuming. A convenient mathematical solution has been developed for a foundation consisting of a single horizontal pervious layer underlying an impervious blanket (6). The range of accuracy of the solution resulting from the simplifying assumptions, is well within the range of accuracy in determining the coefficients of permeability of the foundation layer and the impervious blanket. This solution is presented in Figure 86.

For small dams, a more simplified approach may be adequate to establish the thickness and length of an upstream impervious blanket. A suitable thickness for small dams is 10 percent of the depth of the reservoir above the blanket, with a minimum blanket thickness of three feet (210). The length of the upstream impervious blanket should coincide with the desired reduction in underseepage, and can readily be established since the amount of underseepage is inversely proportional to the length of the flow path.

No major earth dams were noted to have downstream impervious blankets. Mississinewa Dam in Indiana contains a five-foot thick impervious blanket which was placed over the pervious foundation beneath the downstream portion of the embankment as shown in Figure 87. A two-foot thick horizontal filter was placed over this impervious blanket. This impervious blanket provides a barrier to underseepage into the downstream portion of the embankment. Uplift pressures are reduced by a line of relief wells at the downstream toe (108).

The surface layer overlying pervious foundation soils at most damsites and reservoir areas is less pervious than the underlying sediments. This natural blanket is generally continuous throughout the



Because of leakage, the effectiveness of the blankets upstream and downstream from the dam core is reduced. The effectiveness of such blankets is expressed in terms of effective length:-

$$L_{1\,eff.} = \frac{\tanh(a L_1)}{a}; \quad L_{3\,eff.} = \frac{\tanh(a L_3)}{a}$$

$$\text{Where } a = \sqrt{\frac{k_b}{k_f z_f z_b}}$$

Where k_b = the vertical coefficient of permeability of the blanket.

k_f = the horizontal coefficient of permeability of the pervious foundation. The approximate value of foundation seepage per unit length of structure is:-

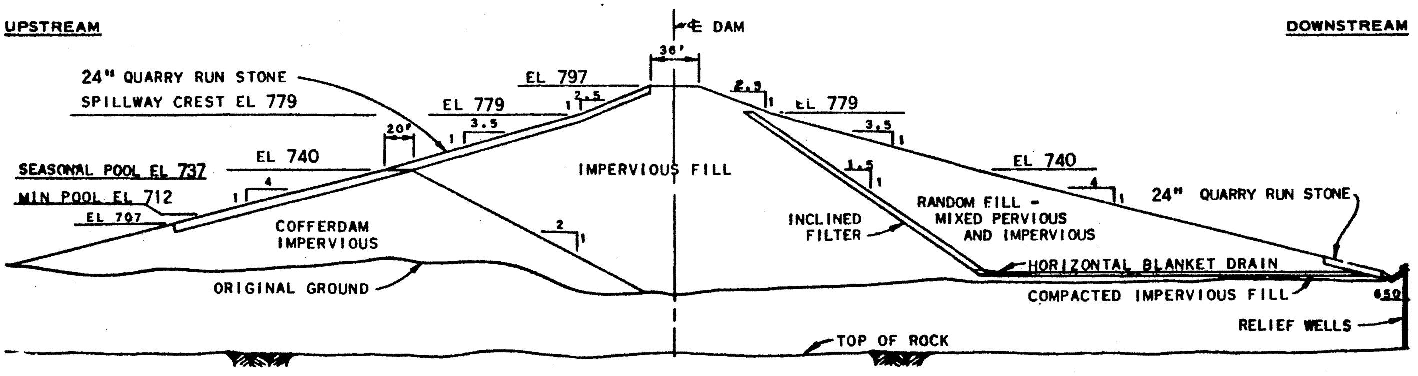
$$Q = \frac{k_f \Delta H z_f}{(L_{1\,eff.} + L_2 + L_{3\,eff.})} = k_f z_f i, \quad \text{Where } i = \frac{\Delta H}{(L_{1\,eff.} + L_2 + L_{3\,eff.})}$$

The piezometric head from D to C is $h_x = \dfrac{I \sinh(a y)}{a \cosh(a L_3)}$; If $L_3 \longrightarrow$ infinity, then $h_x = i a e^{-ax}$

If landside outlet is blocked, then $L_{3\,eff.} = \dfrac{1}{a \tanh(a L_3)}$; $h_x = \dfrac{i \cosh(\alpha y)}{\alpha \sinh(\alpha L_3)}$

Source: (205)

Figure 86. Effectiveness of Upstream and Downstream Impervious Blankets.



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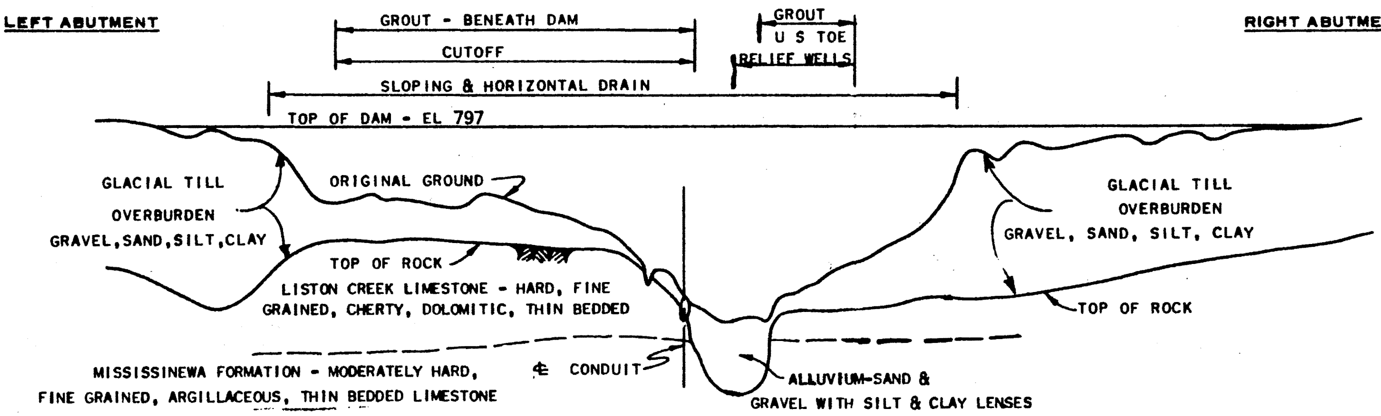


VALLEY CROSS SECTION

Source: (108)

Figure 87. Mississinewa Dam, Mississinewa River, Indiana.

valley floor except where it has been locally removed by stream erosion. The integrity of this natural blanket can be maintained by restricting excavation, particularly borrow excavation, within specified limits upstream and downstream of the embankment. The integrity of the natural blanket is enhanced by backfilling abandoned stream channels and localized erosion ditches with compacted fill.

Summary of Methods

Precluding time and cost constraints, the selection of a method of preventing or reducing underseepage through pervious foundation soils is dependent upon the depth, gradation and stratification of the foundation soils; the characteristic of the underlying rock; availability of suitable on-site materials; and the effectiveness of eliminating or reducing underseepage flow. The best method is a cutoff trench backfilled with rolled, impervious earthfill, if it satisfies the technical constraints, as well as the time and cost constraints. Table XI presents some attributes of the various methods.

Control of Underseepage

Pervious soil foundations with no positive cutoff of underseepage require control devices to relieve uplift pressures within the foundation and to provide erosion resistant discharge of underseepage flow. Control devices may be used in conjunction with partial seepage reduction. Control devices may also serve as a secondary line of defense behind a positive cutoff. These control devices include horizontal filter blankets, toe drains and ditches, relief wells, and foundation drains.

TABLE XI

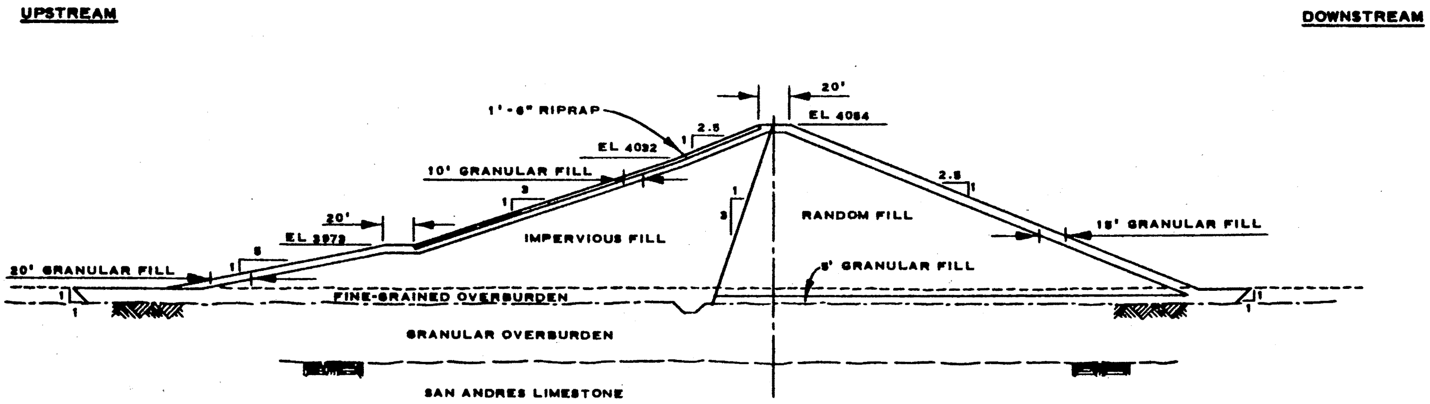
COMPARISON OF TYPES OF CUTOFFS

TYPE	EFFECTIVENESS	FOUNDATION TYPE	ASSURITY	DEPTH FT.	SITE MATERIAL	ADDITIONAL MEASURES
Removal	95 %	any	Excellent	40	-	-
Cutoff Trench	95 %	any	Excellent	75-150	yes	-
Sheet Piling	25 %	soils only	Poor	100-250	no	wells, drains
Concrete Wall	90 %	any	Good	80-250	no	-
Slurry Trench	90 %	soils only	Good	80- 95	yes	-
Alluvial Grouting	90 %	any	Fair	300-680	-	-
Upstream Blanket	33 %	any	Excellent	-	yes	wells, drains

Horizontal Filter Blankets

A horizontal filter blanket intercepts underseepage flow throughout the downstream portion of the embankment foundation, thereby preventing concentrated flow at the toe, and directs the flow to a controlled outlet. An example of a horizontal filter blanket is provided by the typical embankment section in Figure 88 of Rocky Dam in New Mexico. At Rocky Dam, the 10-foot thick surface layer of clay and sandy clay was removed from the downstream portion of the embankment foundation to expose the underlying clayey sandy gravel. A 5-foot thick horizontal filter blanket of granular fill was placed in contact with the exposed clayey sandy gravel to relieve uplift pressures within the foundation which otherwise could have caused a blowout type of failure of the impervious surface layer beyond the downstream toe. The horizontal filter also directs the underseepage flow to a granular-filled toe ditch, and the gradation of the horizontal filter blanket prevents the erosion of fines from the foundation. As seen in the example of Rocky Dam, a horizontal filter blanket must satisfy three requirements (210): (1) gradation must be such that particles of soil from the foundation and the overlying embankment are prevented from entering the filter and clogging it; (2) capacity of the filter must be such that it adequately handles the total seepage flow from both the foundation and the embankment; and (3) permeabililty must be great enough to provide easy access of seepage water in order to reduce seepage uplift forces.

The limits recommended to satisfy graded filter criteria and to provide ample increase in permeability between base and filter are



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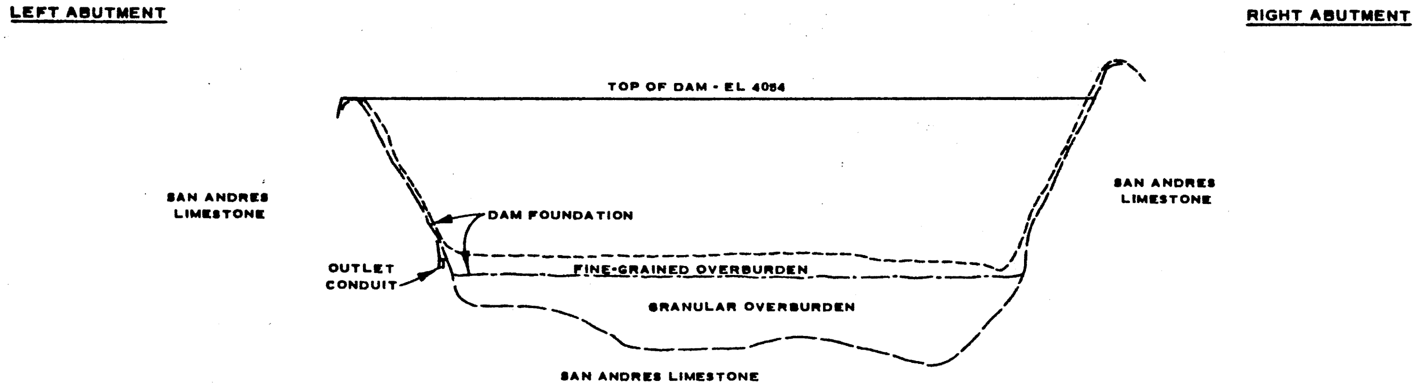


VALLEY CROSS SECTION

Source: (181)

Figure 88. Rocky Dam, Rocky Arroyo, New Mexico.

given in Table II, Chapter 2. Only a slight difference exists between limits recommended by the Corps of Engineers and those recommended by the Bureau of Reclamation. The three-inch particle size should be the maximum utilized in a filter in order to minimize segregation and bridging of large particles during placement. For base materials containing gravel, the gradation of the fraction smaller than the No. 4 sieve should be used in design analyses (210).

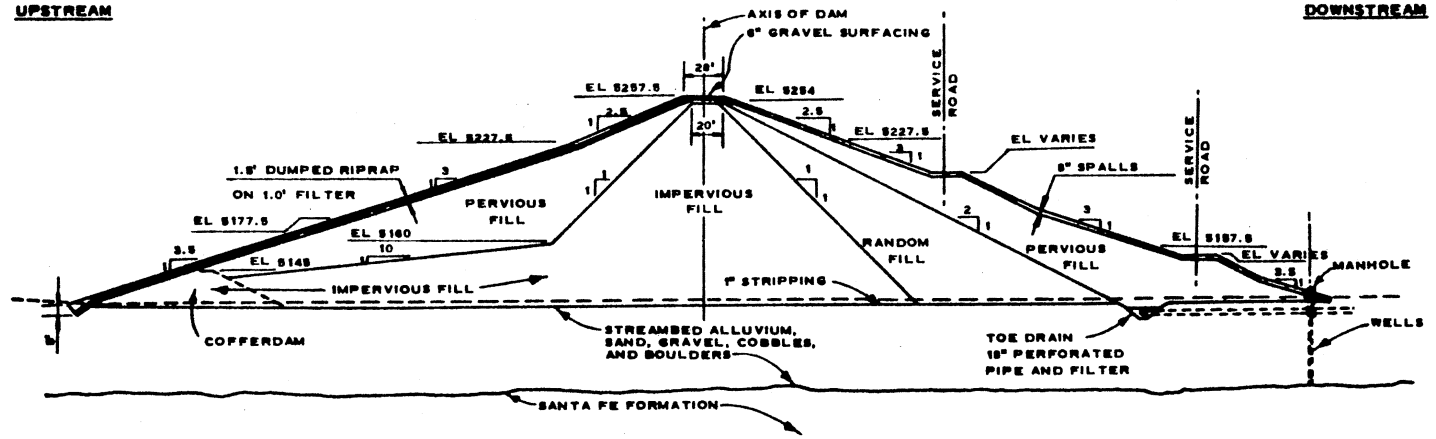
After satisfying the filter criteria, the capacity of selected filter material is dependent on the thickness. Earth dams constructed by the Corps of Engineers since the development of filter criteria contain horizontal filter blankets ranging in thickness from 1.5 feet, at Jadwin Dam in Pennsylvania, to 20 feet, at Harlan County Dam in Nebraska. At Jadwin Dam the horizontal filter blanket is located between the downstream random rockfill zone of the embankment and alluvial zones of silt and sand in the foundation. The horizontal filter blanket was deleted over exposed glacial till in the foundation. At Jadwin Dam the capacity of the horizontal filter blanket is incidental because the blanket functions primarily as a filter or transition zone between the silt and fine sands of the foundation and the rockfill of the embankment. At Harlan County Dam, the foundation consists of loess, which was removed and recompacted, overlying sand and gravel. The homogeneous embankment was constructed of loess, as was the cutoff trench to rock (200). The horizontal filter blanket constructed of sand has a minimum thickness of 20 feet to allow for some contamination by migration of silt (loess) particles into the upper and lower bounds of the filter. The thickness of the horizontal filter blanket is too dependent upon specific site conditions to establish a recommended minimum for general application. However, most earth dams of the Corps of Engineers have blanket thicknesses between 3 and 5 feet.

Several Corps of Engineers dams do not have horizontal filter blankets because the foundations are well-graded sands and gravels. Jerez Canyon Dam in New Mexico, as shown in Figure 89, was not provided with a distinct horizontal filter blanket, but was constructed with a toe drain and relief wells through the valley section.

Usually the type of pervious material used as the horizontal filter blanket is of little concern as long as the gradation satisfies the filter criteria. However, crushed limestone was utilized to construct many horizontal filter blankets and continues to be used. The problem with crushed limestone is that as it weathers in the partially submerged blanket the mass will ultimately become cemented to the point that the filter becomes impervious. This phenomenon has occurred at several earth dams, including Canyon Dam in Texas and Marian Dam in Kansas (209). McGee Creek Dam, presently under construction in southeastern Oklahoma by the Bureau of Reclamation, has a layer of crushed limestone to serve as the horizontal filter blanket.

Toe Drains and Toe Ditches

Toe drains are commonly constructed along the downstream toe of earth dams to intercept embankment seepage and foundation seepage discharging from horizontal filter blankets. The toe drains convey the seepage for discharge into spillways, stilling basins, or stream channels below the dam. Early toe drains were rock filled ditches, or French drains. For the past thirty years new drains have increasingly



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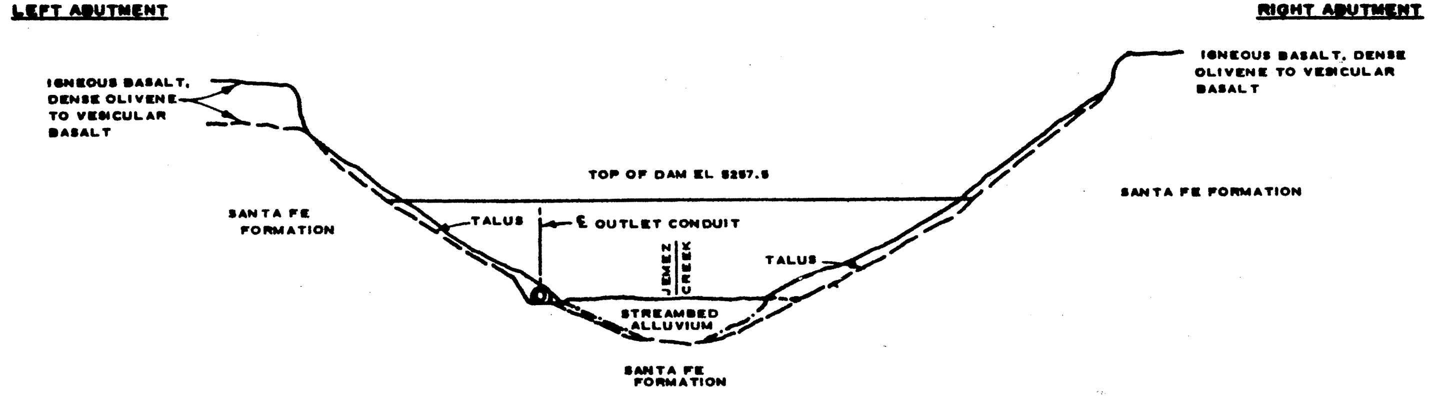


VALLEY CROSS SECTION

Source: (168)

Figure 89. Jemez Canyon Dam, Jemez Creek, New Mexico.

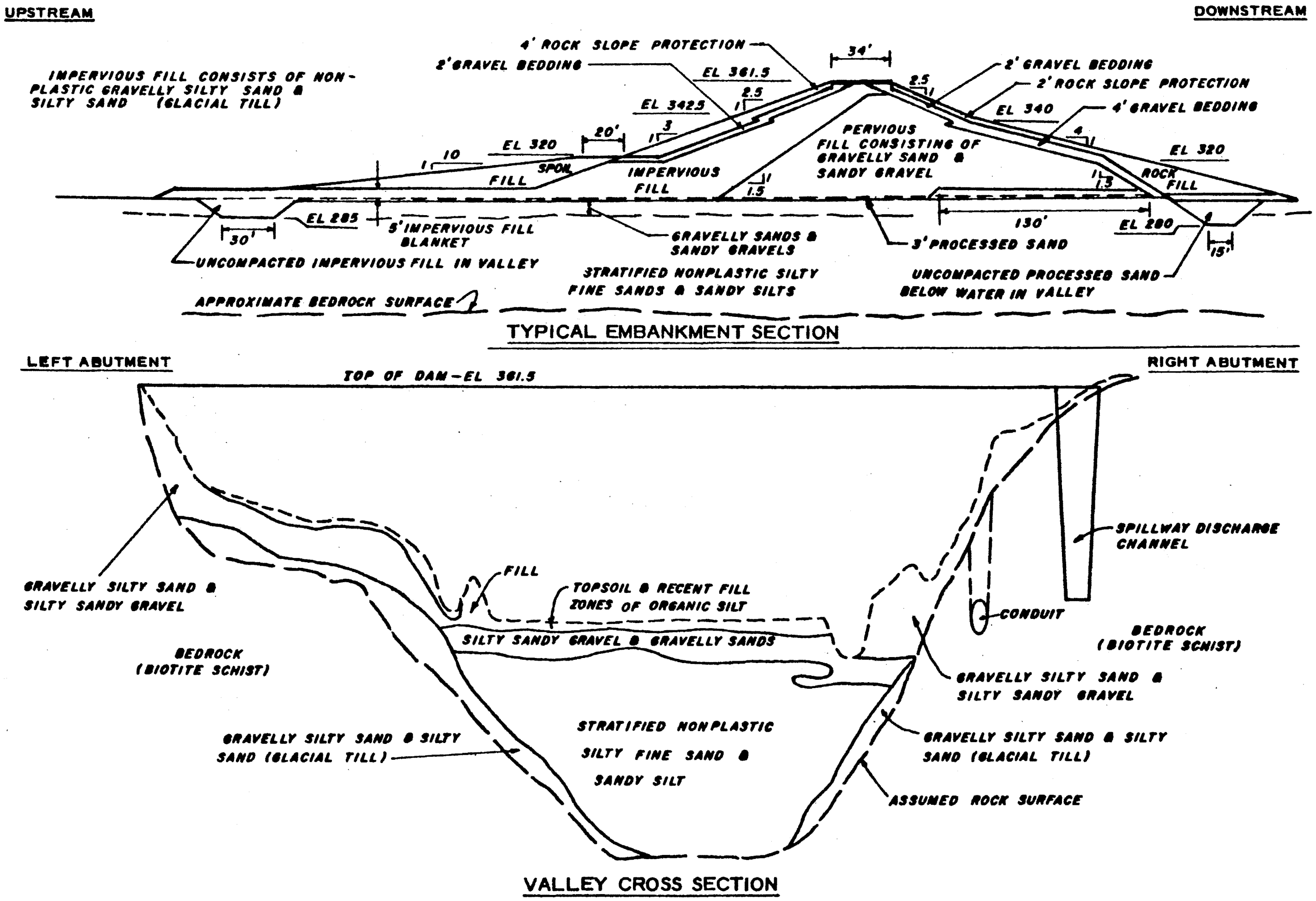
been perforated pipes surrounded by granular fill in trench excavations. The filter criteria for the granular fill is included on Table II, Chapter 2. Figure 90 shows a typical embankment section with sand-filled toe drain at West Thompson Dam in Connecticut. Figure 91 shows the typical embankment section with perforated pipe toe drains at San Angelo Dam in Texas.

Toe ditches are open trenches along the downstream toe of earth dams to intercept embankment and foundation seepage for conveyance to downstream discharge. Toe ditches also channel surface runoff. Toe ditches may be lined or unlined. However, a deep unlined trench requires an inverted, graded filter along the trench bottom to prevent piping. Figure 92 shows the typical embankment section with toe ditch at Okatibbee Dam in Mississippi.

An advantage of a toe drain is the elimination of any visible seepage to reassure the safety of the dam to the public and to engineers who are not conversant with earth dams and seepage. This elimination of visible, or surface, seepage also facilitates maintenance. Cleaning of perforated-pipe toe drains can readily be performed by commercial sewer-cleaning companies with inspection by closed-circuit television.

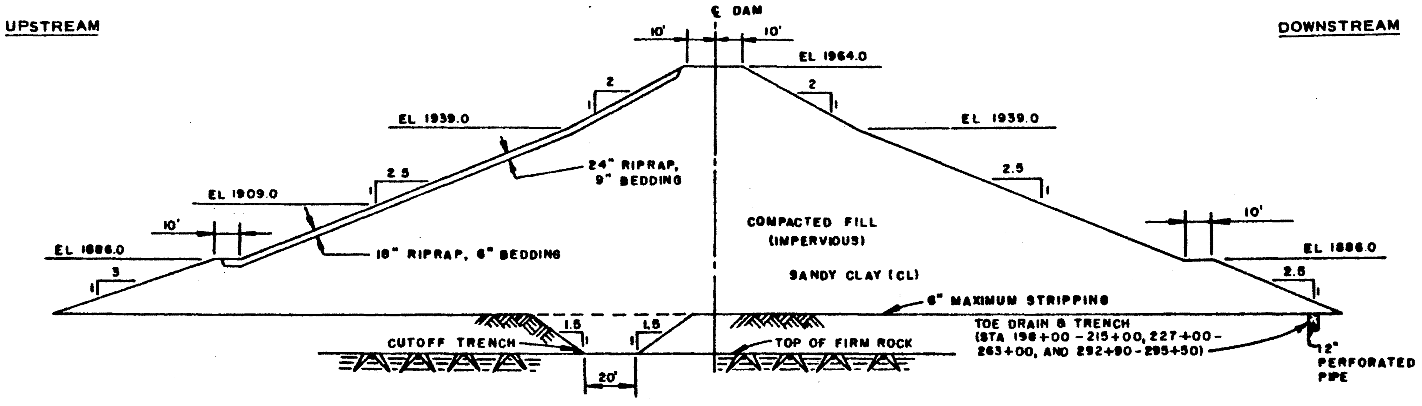
A hazardous disadvantage of perforated-pipe toe drains is that heavier-than-air gases may collect in the manholes. Fatalities have been attributed to natural gas displacing oxygen in manholes at El Dorado Dam in Kansas.

The Corps of Engineers surveyed 41 earth dams in the Southwestern Division. Their evaluation regarding toe drains of dams with positive cutoffs and dams with partial or no cutoffs are presented in Tables XII



Source: (191)

Figure 90. West Thompson Dam, Quinebaug River, Connecticut.



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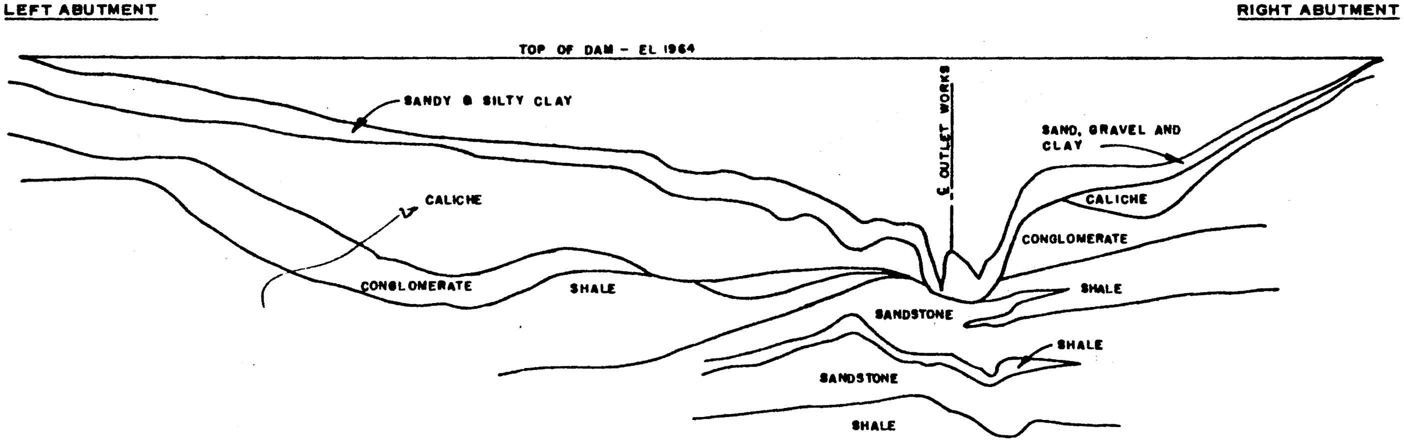
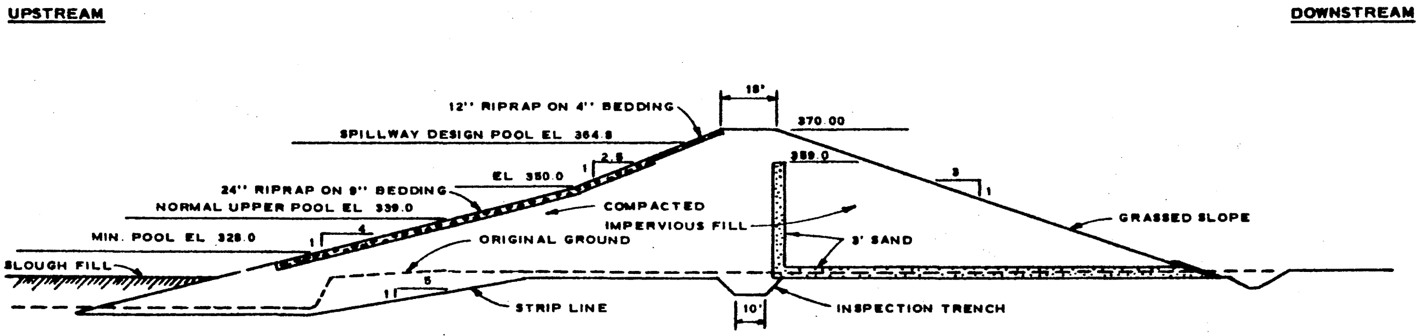


VALLEY CROSS SECTION

Source: (183)

Figure 91. San Angelo Dam, North Concho River, Texas.



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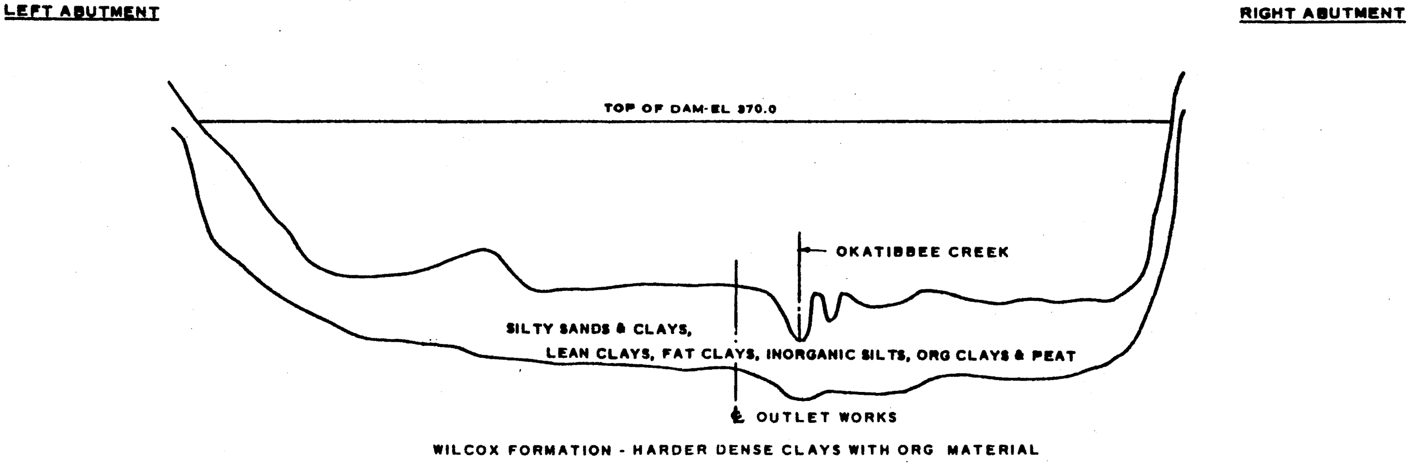


VALLEY CROSS SECTION

Source: (178)

Figure 92. Okatibbee Dam, Okatibbee Creek, Mississippi.

TABLE XII

TOE DRAINS OF DAMS WITH POSITIVE CUTOFFS

DAM	TOE DRAIN	TYPE OF DRAIN	WHEN INSTALLED	NEED
Conchas	yes	Rockfill	Construction	questionable
Belton	no	-	-	no
Benbrook	no	-	-	no
Canyon	yes	Perf. pipe	After constr.	yes
Hoards Creek	yes	Rockfill	Construction	questionable
San Angelo	yes	Perf. pipe	Construction	questionable
Stillhouse Hollow	no	-	-	no
Whitney	no	-	-	no
Beaver	no	-	-	yes
Clearwater	no	-	-	no
Table Rock	no	-	-	no
Broken Bow	no	-	-	no
Eufaula	no	-	-	no
Keystone	no	-	-	no
Oolagah	no	-	-	no
Pine Creek	no	-	-	no
Tenkiller Ferry	yes	Perf. pipe	Construction	questionable

Source: (209)

TABLE XIII

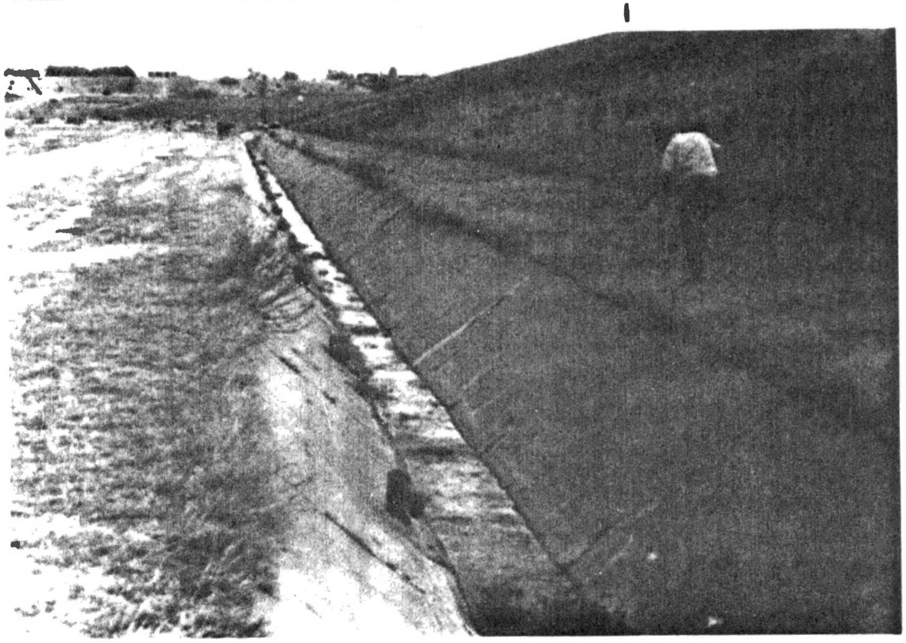
TOE DRAINS OF DAMS WITHOUT POSITIVE CUTOFFS

DAM	TOE DRAIN	TYPE OF DRAIN	WHEN INSTALLED	NEED
John Martin	no	-	-	may
Bardwell	no	-	-	no
Grapevine	no	-	-	no
Lavon	no	-	-	no
Lewisville	no	-	-	no
Navarro Mills	no	-	-	no
Proctor	no	-	-	no
Sam Rayburn	yes	Perf. pipe	After constr.	additional
Somerville	yes	Perf. pipe	Construction	not effective
Waco	no	-	-	no
Canton	yes	Sand trench	Construction	questionable
Council Grove	no	-	-	no
Denison	yes	Perf. pipe	Construction	yes
Elk City	no	-	-	yes
Fall River	no	-	-	may
Fort Supply	yes	Perf. pipe	Construction	questionable
Great Salt Plains	yes	Perf. pipe	Construction	questionable
Heyburn	yes	Perf. pipe	Construction	questionable
Hulah	yes	Perf. pipe	Construction	questionable
John Redmond	no	-	-	may
Marion	yes	Perf. pipe	After constr.	additional
Millwood	no	-	-	no
Toronto	yes	Perf. pipe	Construction	yes
Wister	yes	Perf. pipe	After constr.	questionable

Source: (209)

and XIII, respectively. The significant finding of their evaluation is that in no case was the toe drain essential to the safety of the dam (209).

An advantage of a toe ditch is that, unlike a toe drain, surface runoff can be channeled into a toe ditch along with seepage flow from the embankment and foundation. Relief wells can be installed along the downstream slope of a toe ditch for discharge into the ditch. This allows easy access for maintenance of the relief wells and allows easy observation of individual well flow. The concrete-lined ditch at Great Salt Plains Dam in Oklahoma is shown in Figure 93.



Source: (160)

Figure 93. Concrete-Lined Ditch along Downstream Toe of Great Salt Plains Dam, Oklahoma.

Vandal-proof design has become a most important feature of civil works projects. Exposed relief wells, for example are vulnerable to vandalism. The resistance to vandalism afforded by covered perforated-pipe toe drains, particularly if the pipe also serves as the header pipe for relief wells, is perhaps its biggest advantage over toe ditches. Also, covering or inclosing relief wells can protect the well pipe since polyvinyl chloride and acrylonitrile-butadiene-styrene are now the common materials for relief well risers and header pipes. Both materials are subject to deterioration from the ultraviolet radiation of direct sunlight. The PVC relief wells recently installed in a vandal-prone area below Denison Dam are covered by locked, 18-inch diameter, corrugated metal manholes which are anchored in 12 inches of concrete at the base of the manhole. Figure 94 shows the covered relief wells below Denison Dam. The relief wells are connected to an ABS-truss header pipe which in turn is connected to a corrugated metal manhole with discharge through a grate-covered corrugated metal pipe outlet (149).

Relief Wells

Relief wells installed along the downstream toe of an earth dam relieve the uplift pressures associated with underseepage and provide a controlled outlet for discharge. The less pervious surface and near-surface layers of stratified foundation soils create pressure losses because of their greater resistance to flow (205). The corresponding high uplift pressures may exceed the weight of these layers and result in surface heave. The more likely failure mode, however, is the development of piping of foundation soils through discontinuities in the

Source: (149)

Figure 94. Protected Toe Drain and Relief Well System at Denison Dam.

upper layers, and generally accompanied by boils. Relief wells vertically penetrate the stratified foundation soils to intercept underseepage and relieve uplift pressures within the pervious soil layers.

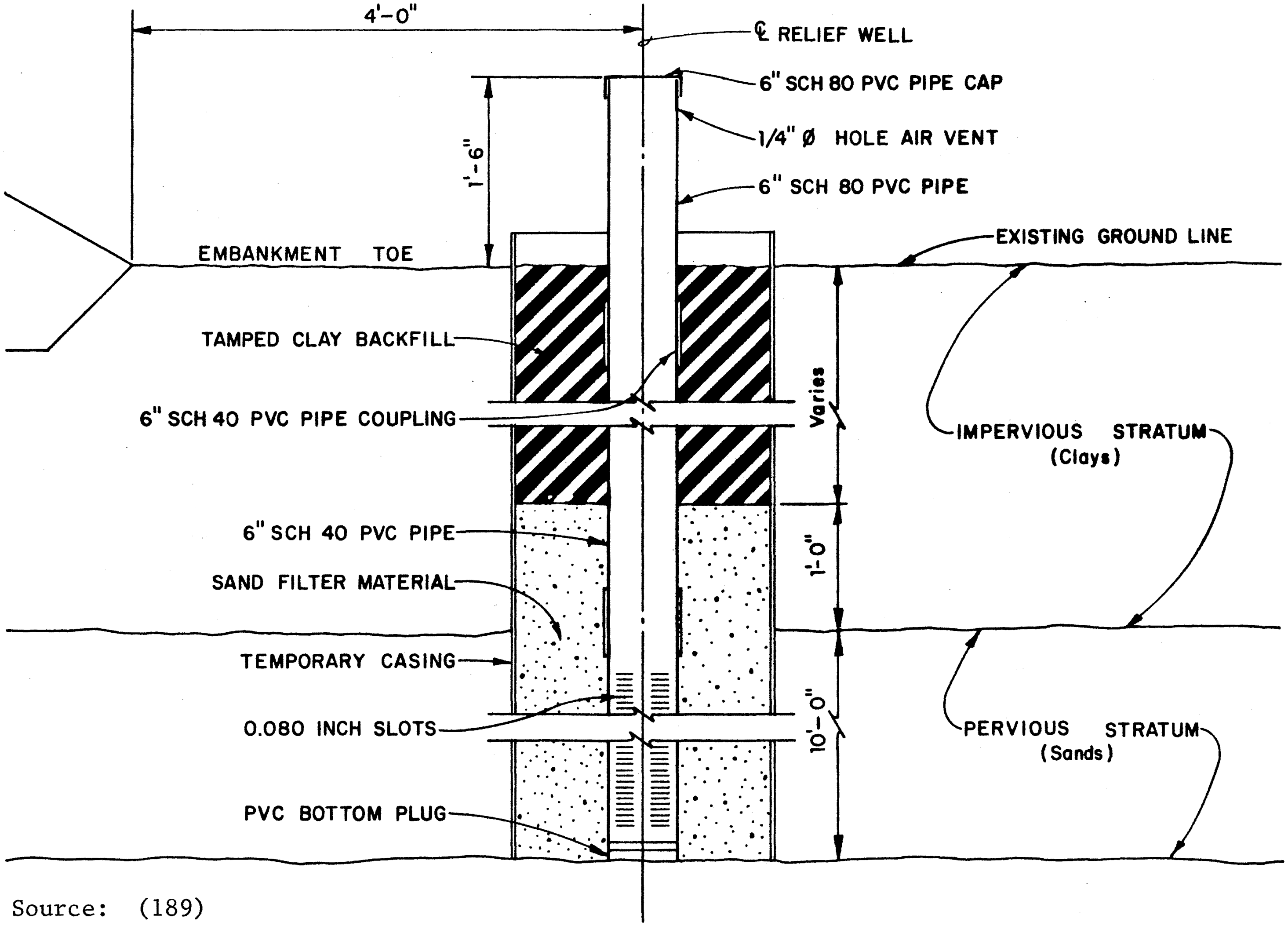
The Corps of Engineers pioneered the development and use of relief wells for seepage control at flood control dams where there is no economic loss connected with underseepage (57). The first application of relief wells to control underseepage resulted from observations of a drilled hole at the downstream toe of the valley embankment section of Fort Peck Dam in Montana. The hole was drilled through the upper clay stratum and penetrated the underlying gravel stratum. Observations when the reservoir was approximately 104 feet above the valley floor indicated a maximum pressure head at the top of the drilled hole equivalent to 45 feet of water. These observations ultimately led to the installation of a system of 21 relief wells along the downstream toe (119).

Engineering manuals are available which present theoretical analyses of underseepage and provide guidance for the design of relief wells (127)(224). However, because of variations within foundation soils, theoretical design is approximate and serves only as a rough guide (57). The limits of location of a relief well system and the spacing between individual wells is most often established through judgment, with prearranged plans to install additional wells if the need should arise. Suggested well spacing is 25 feet minimum for the most pervious foundation to 100 feet maximum for less pervious foundations (210). Model tests performed at the Waterways Experiment Station (Corps of Engineers) indicate that wells which fully penetrate a pervious stratum are twice as effective in reducing uplift pressures within that stratum as wells which penetrate 50% of that stratum and 3 to 4 times as effective as wells which only penetrate 25% (205). The minimum diameter of relief wells should be 8 inches, to provide sufficient slotted, circumferential area for drainage and to provide easier maintenance access and accommodation of submersible lift pumps.

Polyvinyl chloride has replaced wood, bituminous-coated galvanized-metal, and stainless steel as the material for well screens and riser pipes for relief wells. Unlike wood or metal, PVC wells will not rot nor rust in a buried, saturated environment, nor are PVC wells as susceptible to chemical deterioration and scale accumulation. PVC well screens and riser pipes are easy to handle and install, and manufacturers can provide adaptors for proper connections to header pipes or toe drains of different composition. A typical relief well installation using PVC components is shown in Figure 95.

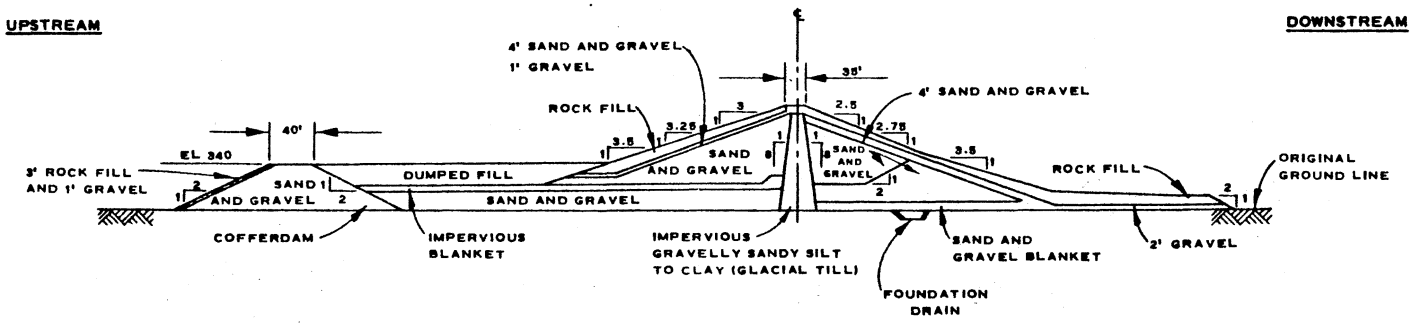
Foundation Drains

Foundation drains below earth dams are gravel-filled trenches or perforated pipe drains located some distance upstream of the downstream toe. Although effective, the decline in installing foundation drains to control underseepage can be attributed to the difficulty in cleaning or replacing deteriorated sections of the drain. A gravel-filled trench as a foundation drain is shown on the typical embankment section of Franklin Falls Dam in New Hampshire in Figure 96. A perforated-pipe foundation-drain was installed at Denison Dam and is located approximately 200 feet upstream of the downstream toe and underlies 70 feet of embankment. The present deteriorated condition of the

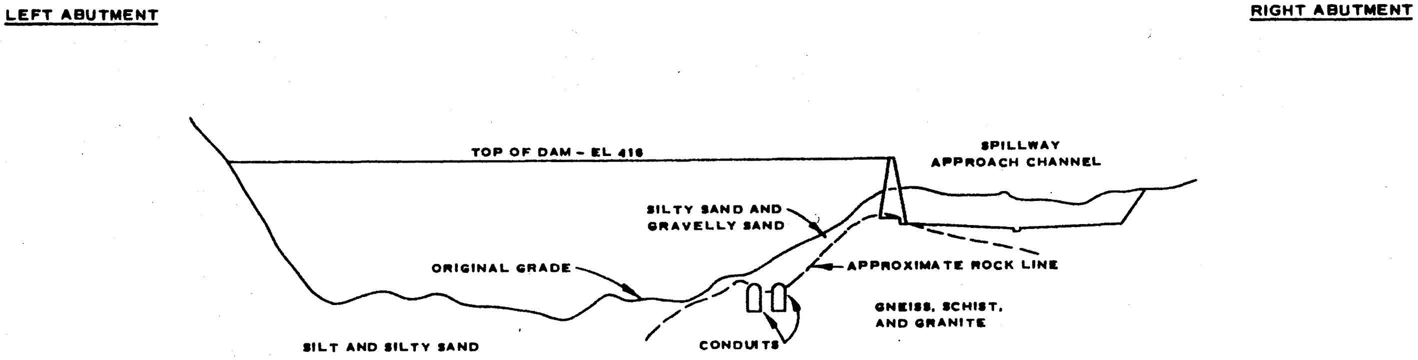


Source: (189)

Figure 95. Relief Well Installation at Waurika Dam, Oklahoma.



TYPICAL EMBANKMENT SECTION

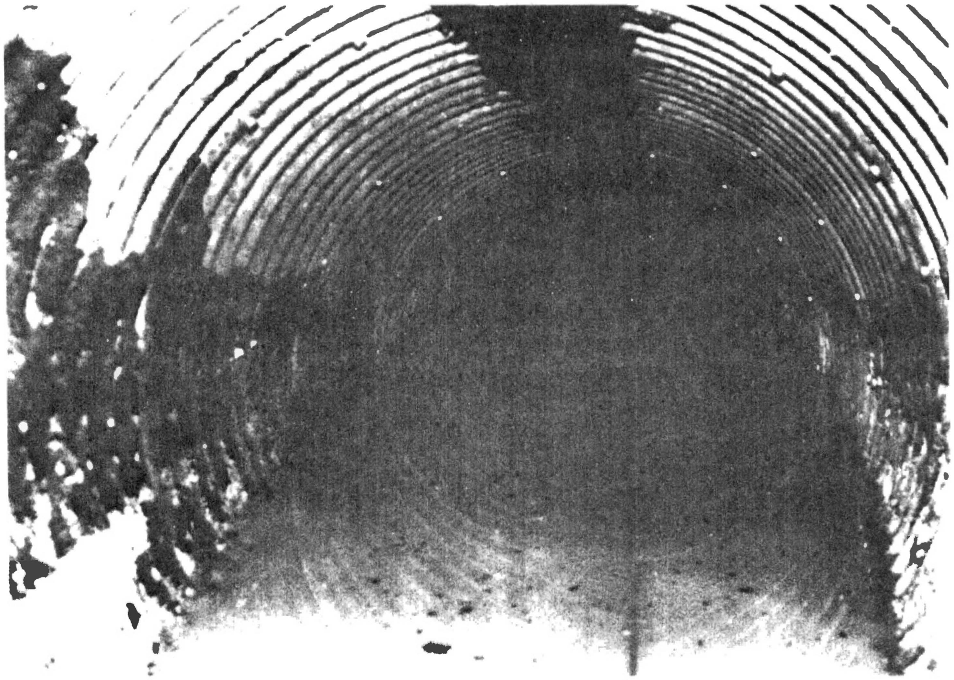


VALLEY CROSS SECTION

Source: (157)

Figure 96. Franklin Falls Dam, Pemigewasset River, New Hampshire.

perforated-pipe foundation drain at Denison Dam is shown in Figure 97. The portion determined to be most deteriorated now has a backup relief well system in the event of failure of that portion of the foundation drain.



Source: (220)

Figure 97. Interior of Foundation Drain at Denison Dam.

CHAPTER V

TREATMENT OF FOUNDATION ROCK

General Design Considerations

The features which distinguish dams from other engineering structures are massive foundation loadings due to great accumulations of construction materials and destructive influences of reservoirs. Consequently, dams depend on the geology of the site more than any other engineering structure. Therefore, the importance of adding geologists to design teams cannot be overstated.

The prominent features of foundation rock which pertain to potential underseepage are the presence of relief joints in valley walls and crushed zones in valley floors. Relief joints are more discernable and result from springing of bedding planes during stream erosion. Crushed zones may not be present to an apprecible extent and result from expansion of the rock during erosion and removal of overlying strata and subsequent release of residual stresses within the rock. The extent of crushing within valley floors is relative to the expansion of the rock through elastic rebound. A small amount of expansion may result in insignificant rupture of the rock, while a large expansion may result in considerable rupture characterized by an easily recognized crushed zone (22)(23). The relief joints in valley walls are always partly filled with loose soil and weathered rock fragments. The crushed zones in valley floors consist of highly fractured rock with soil filling the fractures (25).

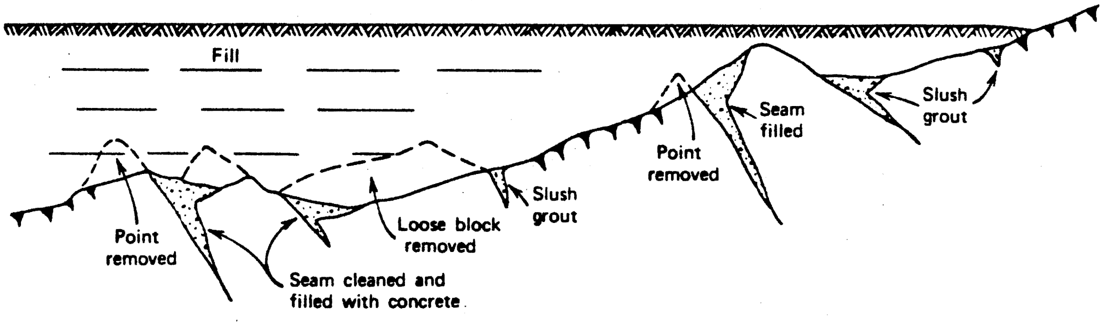
Faults through damsites may provide major conveyance of underseepage flows. The presence within the faults of gouge material and brecciated rock may hinder grouting operations which are implemented to reduce underseepage through faults. Consequently, excavation of the gouge material to a reasonable depth is often necessary, followed by backfilling with lean concrete.

In general, folding does not contribute to the potential of underseepage within foundation rock. However, adequate geologic studies should be completed to determine the effect of areal structural geology on the performance of a potential dam. For example, if the damsite is located over a monocline containing pervious strata, excessive underseepage may occur if the monocline dips downstream (63).

Only within the recent past have geotechnical engineers addressed the problem potential inherent in foundation rock below earth dams. Sadly, the realization of potential problems has occurred through dam failures which ranged from the catastrophic failure of Teton Dam in Idaho to the averted failure of East Branch Dam in Pennsylvania. These failures resulted from neglecting to implement what is now considered to be fundamental treatment of foundation rock. The failure of both Teton Dam and East Branch Dam resulted from piping of earthfill through openings in inadequately sealed rock joints and fractures adjacent to the cutoff trench (29) (104). The two major purposes of treatment of foundation rock are: (1) to provide effective bonding between the impervious zone of the embankment and underlying foundation rock; and (2) to reduce and control underseepage through foundation rock.

Bonding of Impervious Zone to Foundation Rock

In order to provide a suitable contact between the surface of the foundation rock and the impervious zone of the embankment, the rock surface must be smooth enough to allow adequate compaction of the initial lift of impervious material. Figure 98 shows the details of foundation preparation. The rock surface should be cleaned by high-pressure water nozzles or air jets to fully expose the surface for inspection and mapping of significant cracks, as shown for Carters Dam in Figure 99. All rock fragments and rock projections should be removed. All cracks, joints and surface depressions should be filled with dental concrete, slush grout, or shotcrete.



Source: (61)

Figure 98. Details of Foundation Preparation.

Source: (115)

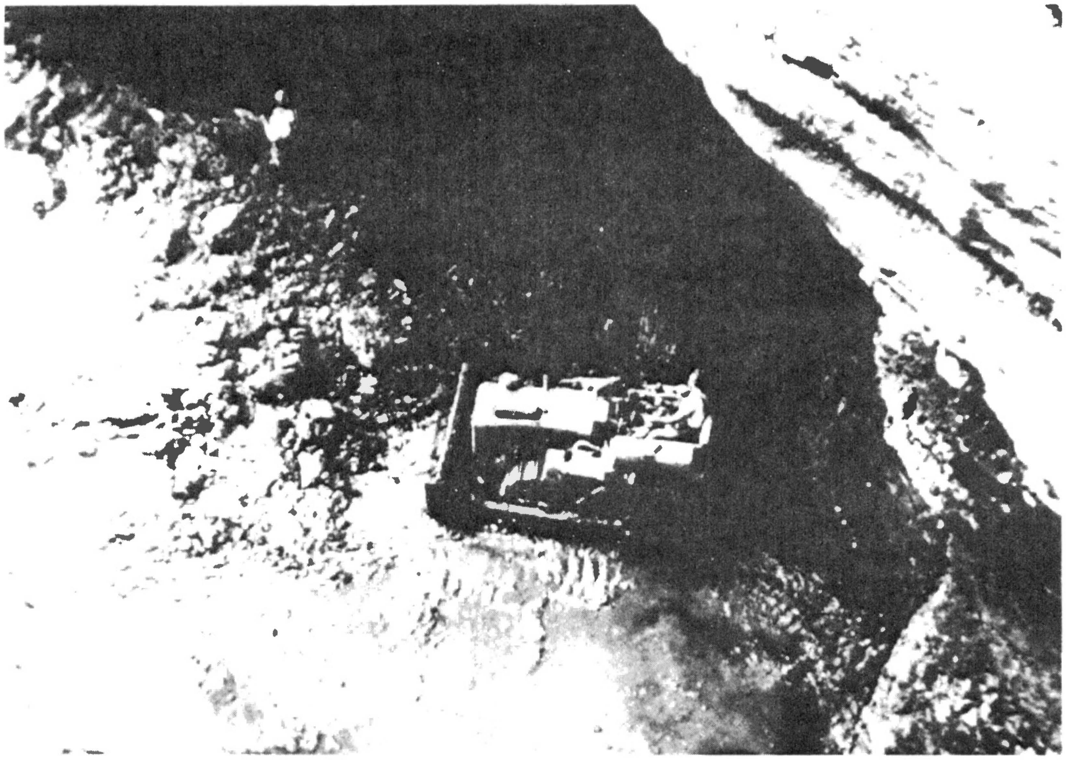
Figure 99. Foundation Cleaning at Carters Dam, Georgia.

Overhangs should be removed to eliminate their interference with adequate compaction of impervious material. Overhangs left in place within the cutoff trench at Wolf Creek Dam, Figure 100, contributed to the massive underseepage problems experienced at that dam (28). Near-vertical abutments should be excavated to more gentle slopes for better compaction and to reduce cracking in the embankment core. High cliffs at the left abutment of Black Rock Dam in Connecticut were flattened approximately to a 1V-on-1H slope (113). The maximum slope of abutments for adequate contact with clay embankment cores is about 70 degrees (128). At damsites where excavation is not feasible to reduce the abutment slope, placement of reinforced concrete, as shown in Figure 101, will adequately provide both reduction in abutment slope and backfill of localized depressions.

Considerations regarding the design of the dam embankment can also contribute to suitable contact with the foundation rock. The width of thin impervious cores should be increased near the rock foundation to provide a wider contact. This is especially advantageous for thin inclined cores because their location, some distance from the axis of the embankment, does not provide maximum contact stresses with the foundation rock. Increasing the width of downstream filters near the rock contact can provide an adequate second line of defense should seepage occur.

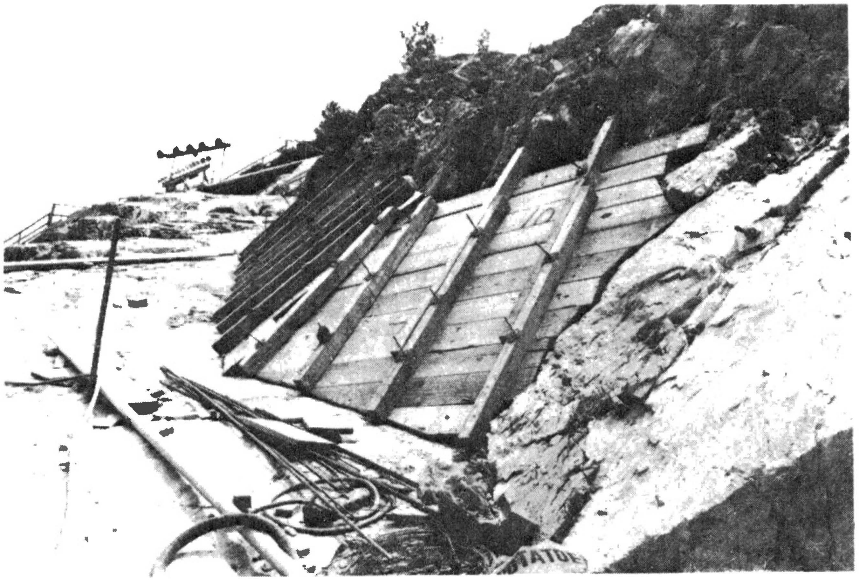
Reduction and Control of Underseepage

Generally, the soundness of rock is such as to preclude erosion by underseepage. However, reduction and control of underseepage in rock may be necessary to prevent erosion of overlying foundation soils and

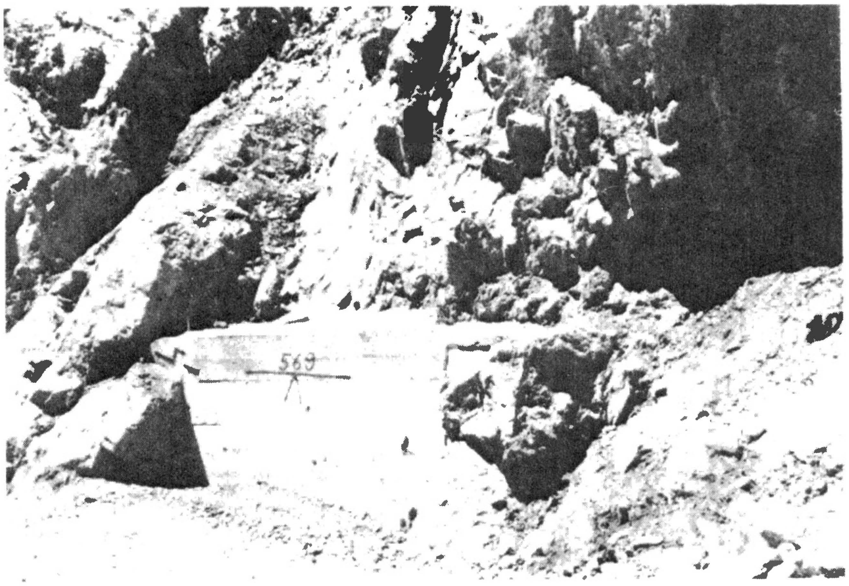


Source: (28)

Figure 100. Overhang Retained in Cutoff Trench of Wolf Creek Dam.



(a) Formwork for Lean-Mix Concrete Placement.



(b) Lean-Mix Concrete in Rockfill Zone.

Source: (122)

Figure 101. Abutment Treatment at New Melones Dam.

embankment fills. Treatment includes grouting, cutoff trenches, concrete cutoff walls and drainage tunnels.

Grouting

Grouting is the injection under pressure of a cementitious material into underlying foundation rock in order to significantly reduce the permeability of that rock. Improving the stability of the foundation rock below an earth dam is considered neither a primary purpose nor an additional benefit of grouting.

The major geotechnical consideration for grouting is to reduce underseepage pressures within foundation rock below the downstream portion of the embankment which might otherwise adversely influence the performance of the earth dam (57). Subsurface conditions which warrant consideration for treatment by grouting include cavernous rock or evidences of solution activity, prominent open joints, broken or intensely jointed rock, faulting, losses of circulation or dropping of drill rods during exploratory drilling, or unusual groundwater conditions (133). An important consideration to be included in determining the necessity of grout treatment is the decrease in permeability of the foundation rock with depth. Most rock types exhibit a definite increase in permeability near the surface due to considerable cracking and joint opening as a result of weathering and stress relief. Therefore grouting can be effective by providing a positive cutoff through the upper zone of higher permeability. However, in some types of rock, such as columnar basalt and some limestones, cracks do not become appreciably smaller with depth and the effectiveness of even very deep grout curtains is minor, and analogous to partial cutoffs in pervious foundation soils (57).

As the availability of materials influences the design of the embankment, the availability of filter materials to construct a horizontal filter blanket might be a consideration for grouting. A local source of abundant pervious materials suitable for use as a horizontal filter blanket may justify deletion or reduction of grout treatment in favor of substantially increasing the blanket thickness. Conversely the expense of long haul routes for the delivery of filter material to provide an adequate blanket thickness might influence a decision to provide grouting as a compensation for reducing the filter thickness. Deep overburden, without provisions for a cutoff trench, eliminates the need for grouting within the underlying foundation rock. Underseepage and related uplift pressures may then be controlled within the overburden. An additional purpose of grouting, and always a benefit, is to provide detailed exploratory drilling of the foundation rock (122).

Table XIV presents 75 earth dams constructed by the Corps of Engineers since 1940 with either removal of overburden or a positive cutoff trench for the entire lengths of the embankments. The significance of these dams is that foundation rock, for the most part, was exposed, thereby providing ready access for installing a grout curtain. Only 45 dams, or 60 percent, have continuous grout curtains beneath the embankments. The predominant type of rock grouted below the dams is sandstone which readily fractures as a result of stress relief or weathering. The least grouted type of rock is shale which tends to swell during stress relief and weathering.

Portlant-cement grout is used exclusively for treatment of foundation rock by grouting. Occasionally, fillers such as sand and fly ash

TABLE XIV

EARTH DAMS WITH EXPOSED TOP OF ROCK FOR FULL LENGTH OF EMBANKMENT

Name of Dam		Rock Type	Reservoir Head	Depth (Primary)	Grout Lines	Remarks	Reference
Abiquiu, New Mexico	1963	Sandstone, mudstone	302	140 max.	1		(82)
Ball Mountain, Vermont	1961	Glacial till	230	-	-	Abutments only.	(79)
Bear Creek, California	1955	Schist	71	-	-		(138)
Belton, Texas	1954	Shale	161	-	-		(90)
Benbrook, Texas	1952	Limestone, shale	93	-	-		(199)
Birch, Oklahoma	1977	Shale, sandstone	76	50	1		(140)
Black Butte, California	1963	Sandstone, conglomerate	99	75	1	Attempted grouting of gravels.	(95)
Black Rock, Connecticut	1970	Quartz-mica schist	134	40	2		(113)
Broken Bow, Oklahoma	1969	Novaculite, shale	168	150 max.	1	Cutoff trench in rock.	(141)
Buckhorn, Kentucky	1960	Sandstone, shale, coal	128	30	3		(75)
Buford, Georgia	1957	Granite gneiss	175	50	1		(91)
C.J. Brown, Ohio	1973	Glacial till	55	-	-		(117)
Cagles Mill, Indiana	1952	Limestone	121	30	1		(142)
Canyon, Texas	1964	Limestone, shale	193	100 min.	1	Additional lines at fault.	(98)
Carlyle, Illinois	1967	Siltstone, shale, sandstone	46	-	-	Abutments only.	(143)
Carters, Georgia	1976	Quartzite, phyllite, angillite	402	75	1		(115)
Clearwater, Missouri	1948	Limestone	113	40	1	Silt added to grout for bulk.	(65)
Cold Brook, South Dakota	1953	Limestone, sandstone	101	40	1		(118)
Colebrook River, Connecticut	1969	Gneiss, granite	194	55	2		(114)
Conant Brook, Massachusetts	1966	Schist	71	25	2		(145)
Coralville, Iowa	1958	Limestone	65	20	1		(76)
Cottonwoods Springs, S. Dakota	1969	Shale, siltstone	104	-	-	Deep cutoff trench at right abutment.	(112)
Cougar, Oregon	1964	Mudstone, tuff	392	220 max.	3		(92)
Coyote Valley, California	1959	Older Alluvium	145	-	-		(70)
Crooked Creek, Pennsylvania	1940	Shale	115	60	1	Not in left abutment.	(146)
Curwensville, Pennsylvania	1965	Sandstone, siltstone, shale	102	75	1	3 lines in fractured sandstone.	(100)
Deer Creek, Ohio	1968	Dolomite	44	40	2	No grouting in abutments - till.	(148)
East Branch, Connecticut	1964	Gneiss	76	25	2		(151)
Eau Galle, Wisconsin	1969	Siltstone, sandstone	103	40	2		(110)
El Dorado, Kansas	1981	Limestone	83	110	1		(152)
Eufaula, Oklahoma	1964	Shale	67	-	-	Abutments only.	(154)
Everett, New Hampshire	1962	Quartz	98	-	-		(80)
Ferrells Bridge, Texas	1958	Indurated sands and clays	70	-	-		(156)
Francis E. Walter, Pennsylvania	1960	Quartzitic sandstone	210	50	1		(84)
Garthright, Virginia	1979	Limestone	196	150	3	Concrete wall in left abutment.	(10)
Grapevine, Texas	1952	Shale, sandstone	109	-	-		(158)
Grayson, Kentucky	1967	Sandstone	96	50	1		(159)
Hall Meadow Brook, Conn.	1962	Gneiss, schist	54	30	2		(161)
Hancock Brook, Connecticut	1966	Schist	36	-	-	Abutments only.	(162)
Harlan County, Nebraska	1952	Chalk	69	-	-		(200)
Hidden, California	1975	Granite	163	-	1		(163)
Howard A. Hanson, Washington	1962	Andesite, basalt	178	30	1		(83)

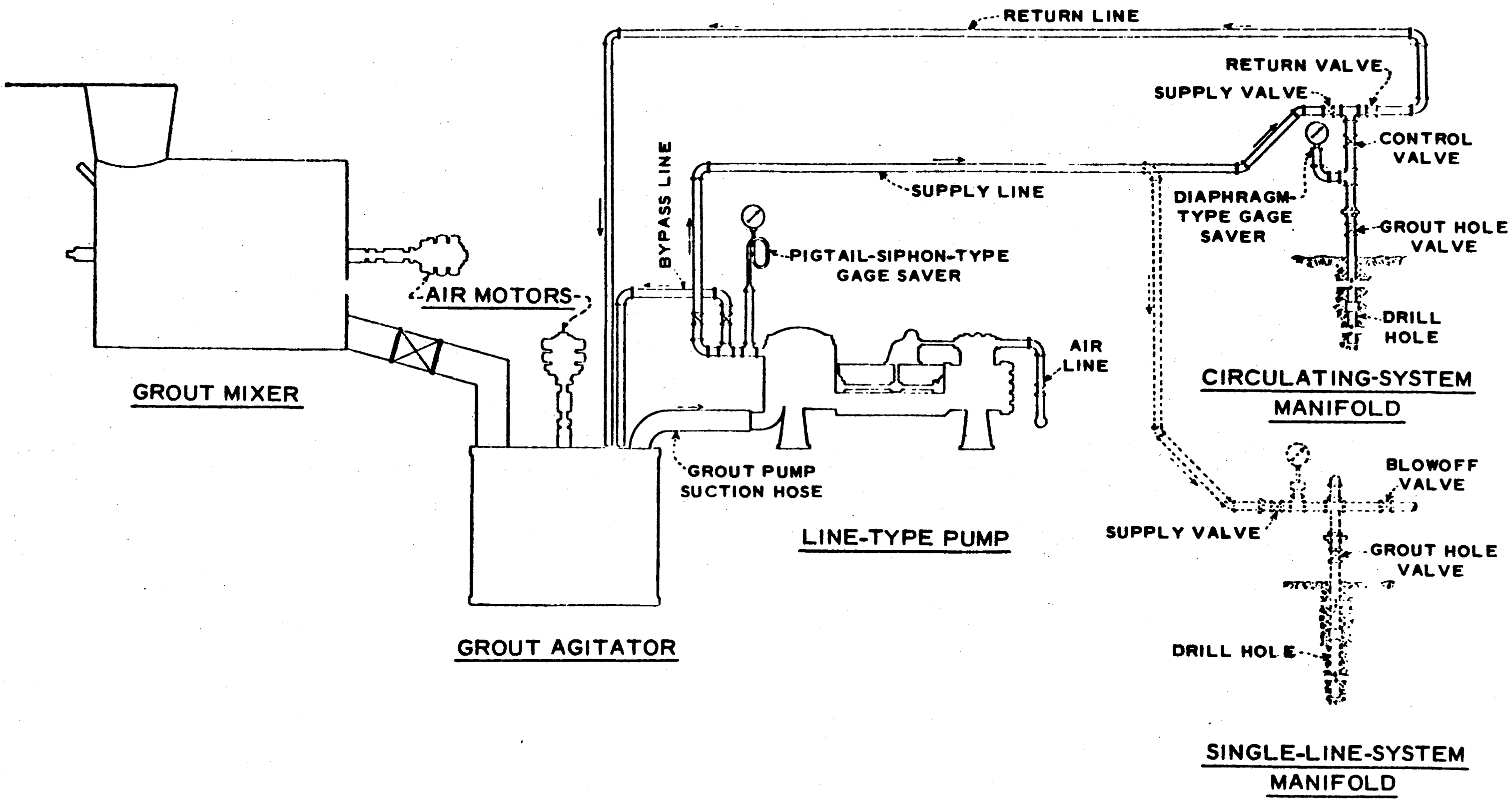
TABLE XIV (Continued)

Hills Creek, Oregon	1961	Andesite	273	-	-	Slush grouting at core contact.	(101)
John Redmond, Kansas	1964	Shale, limestone	36	-	-		(169)
John W. Flannagan, Virginia	1962	Sandstone, shale	250	75	3		(81)
Kaneohe-Kailua, Hawaii	1980	Tuff, basalt	56	-	-		(120)
Keystone, Oklahoma	1964	Shale	69	-	-	Abutments only.	(171)
Lewisville, Texas	1954	Shale	97	-	-		(135)
Littleville, Massachusetts	1964	Glacial till	144	-	-	Abutments only.	(96)
Lookout Point, Oregon	1954	Tuff, breccia	220	-	-		(86)
Lucky Peak, Idaho	1955	Granite, basalt	323	-	-	Left abutment only.	(85)
Mad River, Connecticut	1963	Schist, gneiss	165	30	2		(93)
Melvern, Kansas	1973	Shale, limestone	102	50	1		(121)
New Hogan, California	1964	Sandstone, volcanics	154	75	1		(89)
New Melones, California	1978	Meta-volcanics	588	200 max.	3		(122)
Nolin River, Kentucky	1963	Sandstone, limestone, shale	148	50	2		(97)
Northfield Brook, Connecticut	1964	Quartz-mica schist	103	40	2		(176)
North Fork, Virginia	1966	Sandstone, siltstone, shale	96	60	2		(99)
North Hartland, Vermont	1961	Phyllite, schist	159	-	-		(87)
Painted Rock, Arizona	1960	Basalt, rhyolite, tuff	137	60	1		(73)
R.D. Bailey, West Virginia	1977	Sandstone	280	-	-	Left abutment to 150 feet.	(136)
Sardis, Oklahoma	1982	Shale, Sandstone	63	100	1		(184)
Stillhouse Hollow, Texas	1968	Limestone, shale	168	-	-	Left abutment only.	(109)
Stockton, Missouri	1970	Dolomite	102	60	1		(111)
Success, California	1961	Older Alluvium	103	-	-		(94)
Salamonie, Indiana	1967	Limestone	113	50	1		(182)
Tenkiller Ferry, Oklahoma	1952	Sandstone, shale	162	60	2		(186)
Thomaston, Connecticut	1960	Granite	119	-	-		(78)
Tionesta, Pennsylvania	1941	Shale, sandstone	125	70	1		(187)
W. Kerr Scott, North Carolina	1968	Schist	116	20 min.	1	To sound rock.	(88)
Walter F. George, Georgia	1963	Limestone	88	50	1		(188)
Warm Springs, California	1983	Shale, sandstone, conglomerate	295	200	1	3 lines in right abutment.	(123)
Whitlow Ranch, Arizona	1960	Rhyolite	116	-	-	Abutments and streambed only.	(192)
Whitney, Texas	1951	Limestone	112	-	-	Shallow grouting of foundation.	(193)
Wolf Creek, Kentucky	1950	Limestone	135	75	1		(28)

are added to provide bulk for filling large voids more economically. Mixing and circulation of grout is accomplished by equipment similiar to that shown in Figure 102.

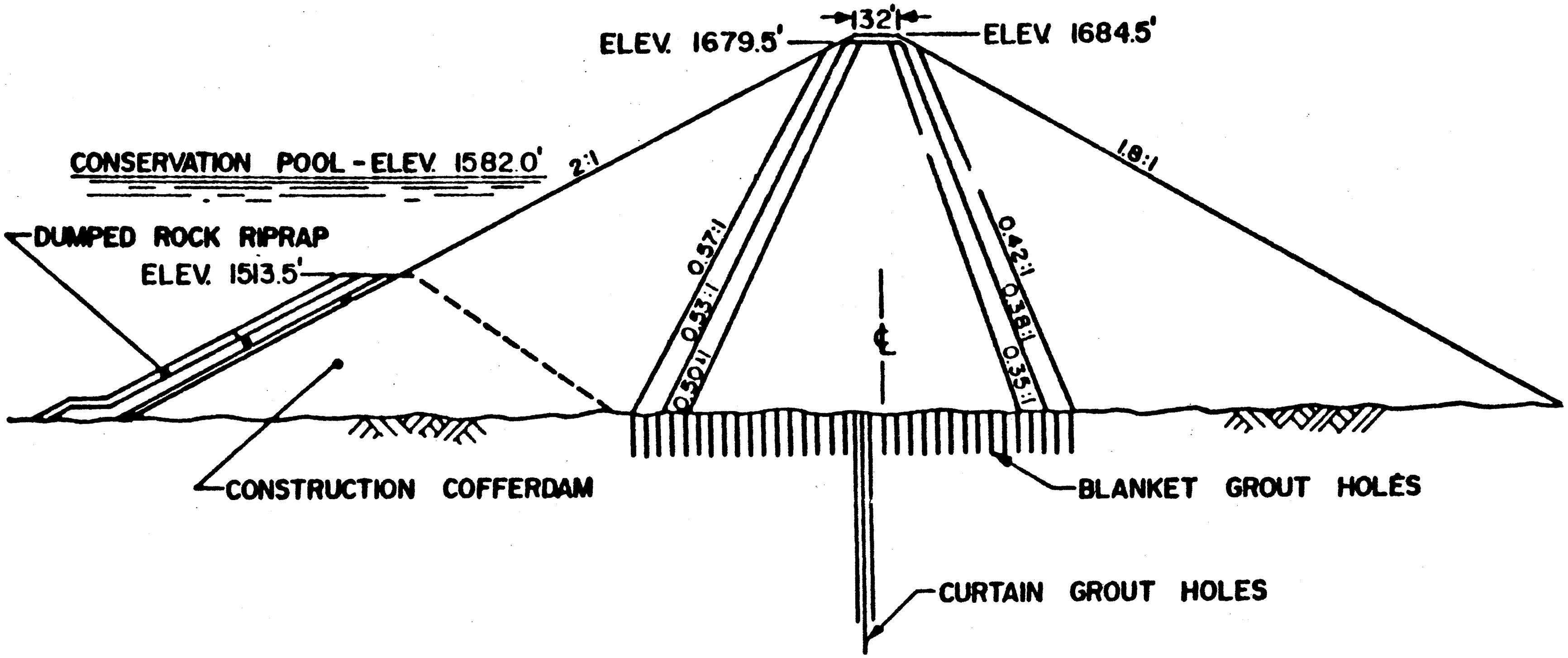
Three types of grouting programs may be considered for treatment of foundation rock underlying a damsite. Curtain grouting is the construction of a vertical or inclined barrier within deep foundation rock. Construction involves progressively drilling and grouting a linear sequence of holes which may be in a single row or in parallel rows. Blanket or consolidation grouting involves drilling and grouting shallow holes, usually on a grid pattern, over a large area of highly fractured surface rock. Contact grouting is used to grout voids between walls of underground excavations and constructed linings. Curtain grouting and blanket grouting for Garthright Dam in Virginia are shown in Figure 103.

The problem with grouting as a treatment of foundation rock is that the voids cannot be completely filled with grout. Initially the voids are partially filled with soil and rock fragments that are not replaced with grout; grout fills only the open voids. In time the soil can be washed out by underseepage from the reservoir (25). This phenomenom can best be illustrated by examples from remedial grouting of a dam foundation with known underseepage problems such as have occurred at Walter F. George Dam in Georgia. Sinkholes discovered in the reservoir area during closure construction in 1962 resulted in construction of a grout curtain along the upstream toe. Localized grouting of sinkholes sporadically occurred until 1982 when construction of a concrete-panel cutoff was initiated. Active solution channels, uncovered during excavation for the concrete panels, contained



Source: (133)

Figure 102. Schematic of Grouting Plant.



Source: (10)

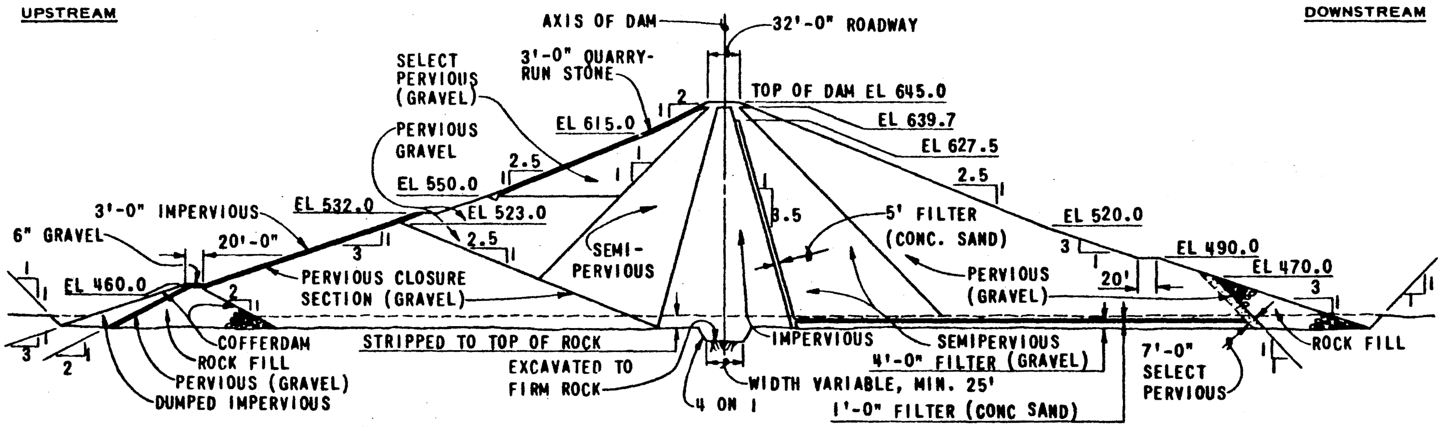
Figure 103. Curtain Grouting and Blanket Grouting at Garthright Dam, Virginia.

highly erodable silts and fine sand in addition to earlier-placed grout (188). Also, the grout holes may not effectively intercept the fractures and joints within a rock mass, thereby leaving openings in the curtain through which underseepage may readily pass. These areas usually correspond to areas of extremely low grout takes during construction of the grout curtain (140).

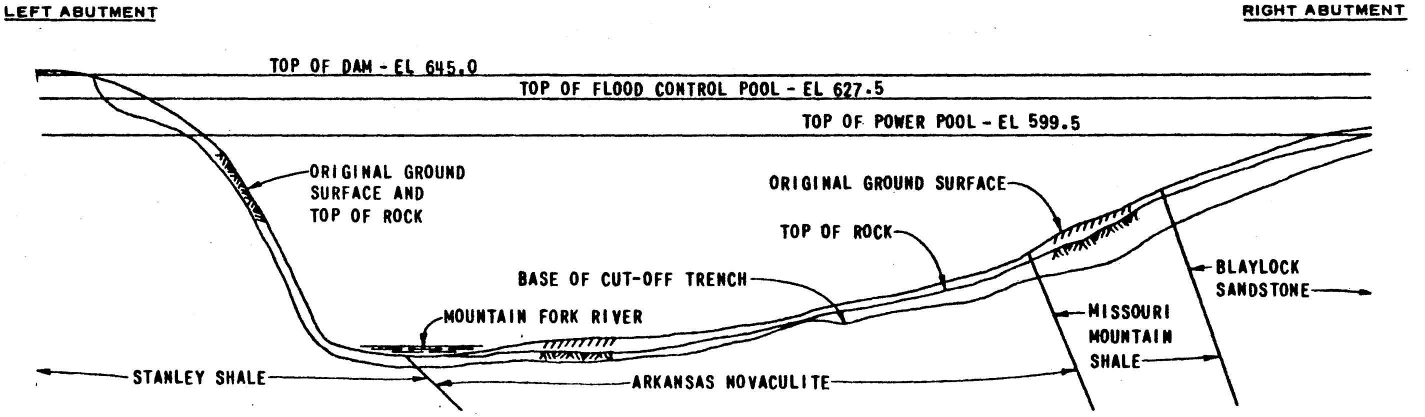
Because of the apparent ineffectiveness of grout curtains to control underseepage through highly fractured or cavernous foundation rock, reliance should not be placed upon grouting to cure the defects in rock (25). Conversely, if seepage problems do not occur, the grouting undertaken as underseepage control could have been unnecessary (57).

Cutoff Trenches

A rolled-earthfill cutoff trenches through fractured and weathered rock is a very effective method to reduce underseepage that might otherwise occur within the near-surface foundation rock. The trench excavation should fully penetrate the surface zone of weathered rock and the width should provide adequate contact with the impervious core of the embankment. A graded filter may be provided along the side slopes of the trenches if appreciable fracturing of the foundation rock or springing of bedding layers is expected to occur as a result of trench excavation. The cutoff trench at Broken Bow Dam in Oklahoma, as shown in Figure 104, was constructed through the entire length of the embankment. At Cottonwood Springs Dam in South Dakota, a rolled-earthfill cutoff trench was constructed only through the cavernous limestone in the right abutment (112).



TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

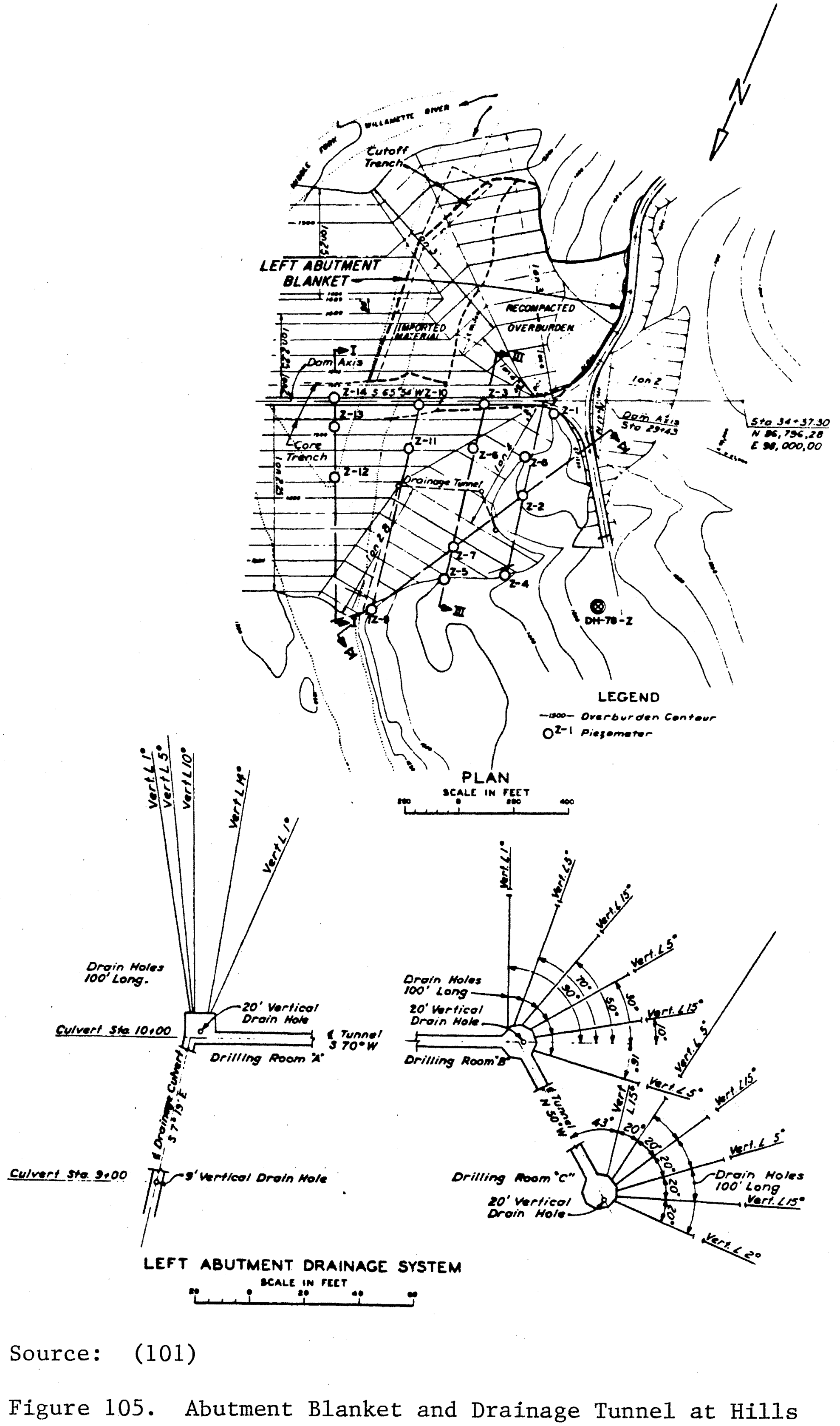
Source: (141)

Figure 104. Broken Bow Dam, Mountain Fork River, Oklahoma.

Drainage Tunnels

Drainage tunnels are effective at intercepting and controlling underseepage in rock, and as an exploratory adit are very effective for discerning the geologic structure and possible inadequacies within foundation rock. Also, drainage tunnels can readily accommodate drilling and grouting equipment to perform remedial grouting from the tunnel or to install additional drain holes. The three earth dams with drainage tunnels constructed by the Corps of Engineers are Hills Creek Dam in Oregon, Garthright Dam in Virginia, and Warm Springs Dam in California.

At Hills Creek Dam the mass structural weakness of the left abutment was produced by an intrusion of an andesitic rock mass which fractured and disrupted the overlying tuff. Regional warping and faulting later fractured the entire mass along the left abutment. Subsequent groundwater percolation in the open fractures resulted in deposition of colloidal clay and in deep weathering of the tuff. A drainage tunnel was driven into the left abutment to intersect the fractured andesitic rock that carried the natural hillside drainage. Drain holes were drilled at low angles from rooms in the tunnel to increase its effectiveness at intercepting seepage. An impervious upstream blanket was placed in order to substantially reduce seepage into the left abutment from the reservoir. The blanket with a minimum thickness of six feet extends 800 feet upstream from the impervious embankment core and covers the abutment to maximum pool elevation. A plan of the drainage tunnel and impervious blanket is shown in Figure 105. A flow of 0.05 cfs from the left abutment drainage system was



Source: (101)

Figure 105. Abutment Blanket and Drainage Tunnel at Hills Creek Dam, Oregon.

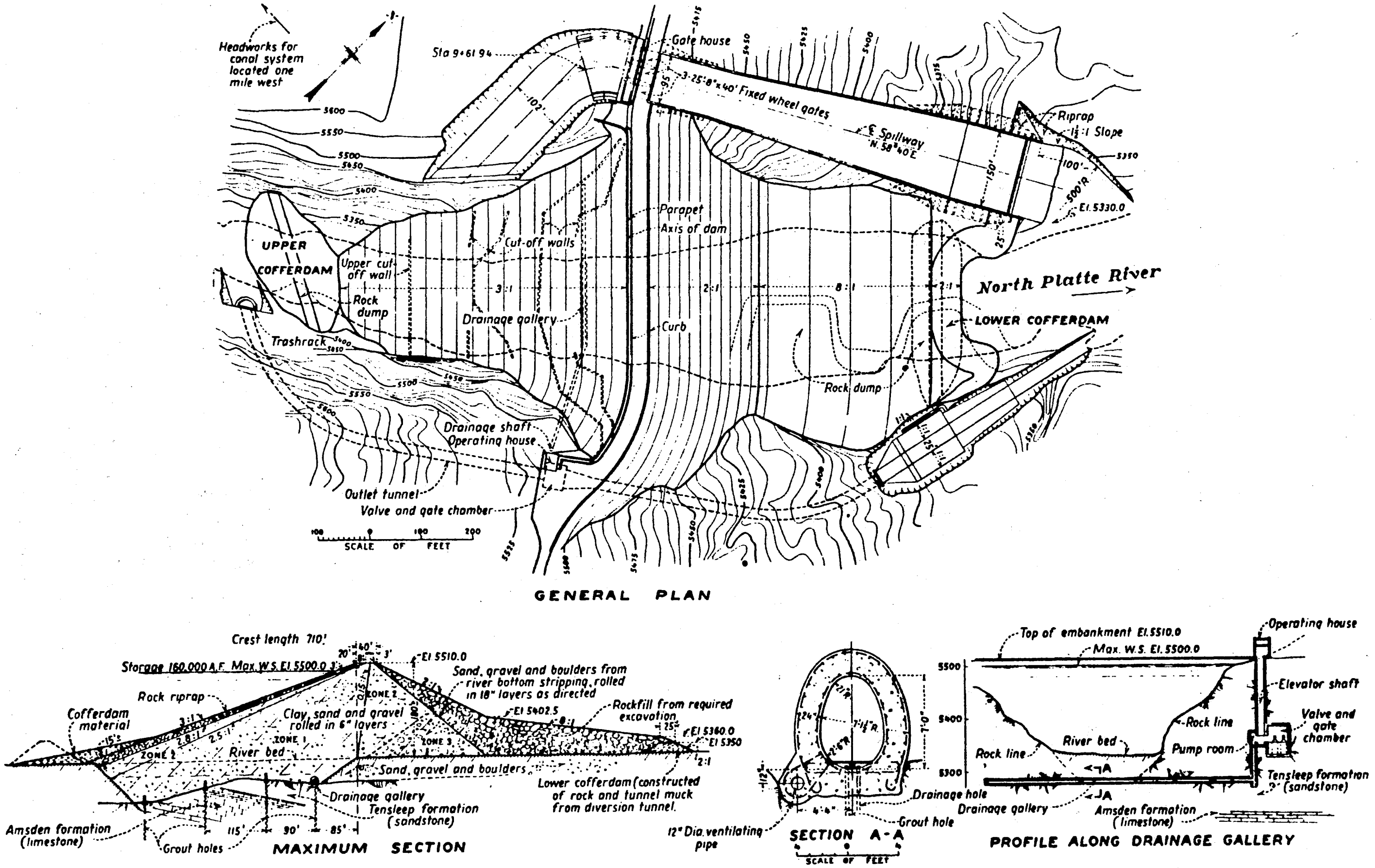
recorded prior to initial reservoir filling in 1962. Flow increased to a maximum of 0.85 cfs at full pool and then decreased gradually. By 1966 the flow decreased to the present flow of 0.35 cfs for maximum reservoir pool. Piezometers installed in the left abutment indicate that the water table at maximum reservoir pool is considerably lower than the preconstruction water table, attesting to the effectiveness of the drainage tunnel (101).

At Garthright Dam in Virginia three adits were driven into the left abutment, with an intersecting adit driven from the downstream portion of the abutment to provide permanent access and downstream drainage. One adit was driven into the right abutment from the downstream portion to provide permanent access and drainage of that abutment. The adits were the result of special foundation investigations which included downhole television cameras and seismic wave front diagrams developed by uphole shooting in some borings. Solutioning was not encountered in the right abutment. However, extensive solutioning was encountered in the left abutment in the form of open caves and widened joints which were partially filled with rock debris and clay. A concrete-membrane cutoff wall was constructed upstream of the adits in the left abutment with the adits intercepting seepage downstream of the cutoff wall (10).

At Warm Springs Dam in California the left abutment is predominantly sandstone which was found to be highly-fractured with open joints and fractures to considerable depths. A drainage tunnel was driven into the left abutment to intercept and remove seepage. The drainage tunnel is 600 feet long with 8 drain holes extending radially from 50 to 150 feet into the abutment every 50 feet of tunnel length.

An access tunnel 810 feet long extends downstream from the drainage tunnel to daylight at a portal at the downstream portion of the abutment (123).

Several earth dams in Europe have either drainage tunnels driven through foundation rock or drainage galleries constructed immediately on the top of rock for the full length of the dam embankment (57). In the United States the use of drainage tunnels has been limited to the abutments and no attempts have been made to drive a drainage tunnel in the foundation rock beneath the valley section of an earth dam. Alcova Dam in Wyoming, constructed in 1935 by the Bureau of Reclamation, has a drainage gallery constructed of reinforced concrete atop the foundation through the valley section of the embankment and accessible through an elevator shaft in the right abutment (211). The gallery is below tailwater elevation and flooded during normal operations; however, a pumphouse was provided to allow entry into the gallery. Drainage galleries were never considered again for Bureau projects because of the great expense related to constructing the gallery at Alcova Dam (215). In operation as a collector of underseepage, the drainage gallery is very similar to the perforated corrugated-metal pipe foundation drain at Denison Dam. Although the operations are similar, the source of underseepage is different. The foundation drain at Denison Dam collects underseepage flowing through a pervious soil foundation while the drainage gallery at Alcova Dam collects underseepage flowing through foundation rock. The embankment plan and typical section of Alcova Dam are presented in Figure 106, as much for the historic novelty as for presentation of a viable alternative for intercepting and controlling underseepage.



Source: (211)

Figure 106. Alcova Dam, North Platte River, Wyoming.

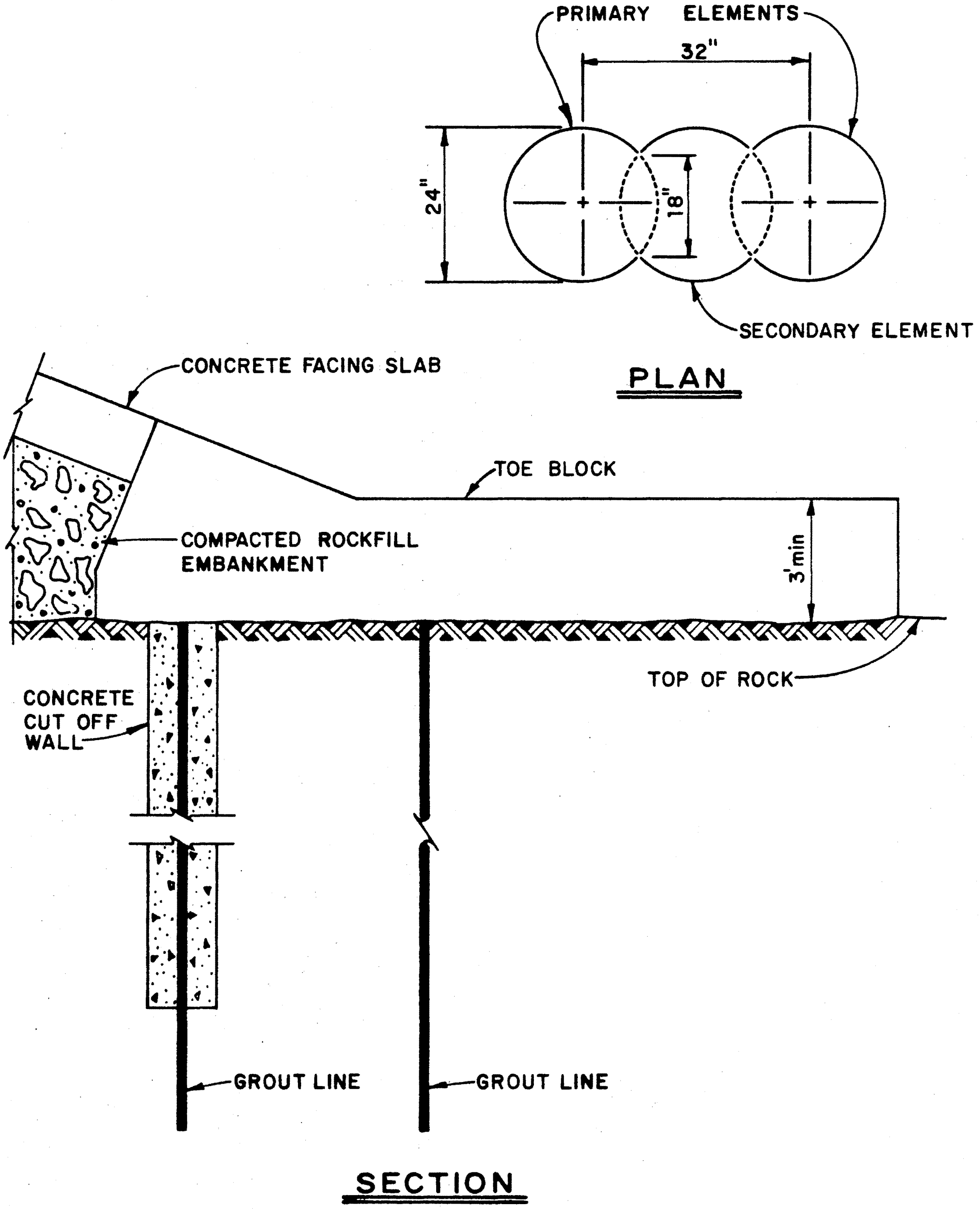
Concrete Cutoff Walls

During the past ten years the construction of concrete cutoff walls has established an important precedent in the United States for the treatment of deep strata of highly fractured or cavernous foundation rock. The best treatment for shallow layers of highly fractured or cavernous rock is to excavate and remove the problem rock from within the areal extent of the impervious core. A shallow layer of highly fractured sandstone at the rock surface was removed under the core and transition zones at East Lynn Dam in West Virginia (25). Within highly fractured or solutioned rock the voids are partially or wholly filled with soil and rock fragments which cannot be replaced with grout. Consequently, a grout curtain cannot be relied upon to prevent underseepage and, with time, the soil can be eroded from the voids by underseepage through the dam foundation. Embankment soil compacted against highly fractured or cavernous rock may be eroded into the voids by underseepage. The best defense against piping by erosion of the soil and rock fragments in voids within foundation rock is to construct an erosion-resistant, impervious membrane through the problem rock in order to provide a waterstop.

The major obstacle to concrete cutoff walls in pervious foundation soils, as well as in embankments, is that the presence of the concrete wall introduces a rigid discontinuity within the compressible soil mass. Obviously this problem is insignificant in foundation rock. In this respect, a concrete cutoff wall in foundation rock is analogous to a slurry trench cutoff in foundation soils. The problems of constructing a slurry trench cutoff in foundation rock are that the reservoir head could drive the slurry backfill into the voids and the inevitable consolidation of the slurry backfill could create a seepage path immediately above the cutoff (223). Arching of the embankment material should be expected to bridge over the narrow trench excavation of the cutoff.

At R.D. Bailey Dam in West Virginia, a crushed zone of severely fractured sandstone underlies the dam embankment from about the valley center to and including the lower portion of the right abutment. The depth of the crushed zone, 60 feet maximum, precluded removal of the fractured sandstone to expose sound rock. The crushed zone was not considered detrimental to the stability of the embankment, and a concrete cutoff wall has been constructed to provide positive reduction of underseepage. The concrete wall was constructed by using a 24-inch diameter hammer to drill the primary holes, and then to drill the overlapping secondary holes after the concrete in the primary holes had set (24). Details of the cutoff wall and its relationship to the toe block for the concrete facing slabs are shown in Figure 107. While the interfacing of the upstream concrete cutoff wall with the concrete slab facing of the upstream slope through the use of a concrete toe block was innovative in scope, a similar treatment was accomplished at McKay Dam in Oregon which was constructed by the Bureau of Reclamation in 1923 (211).

At Patoka Dam in southern Indiana, a 1400-foot long concrete cutoff wall was constructed between the spillway and the embankment to assure retaining a permanent reservoir pool. The cutoff wall is 5.5 feet wide and was constructed to a depth of about 30 feet through cavernous limestone to a relatively impervious shale. Excavation of



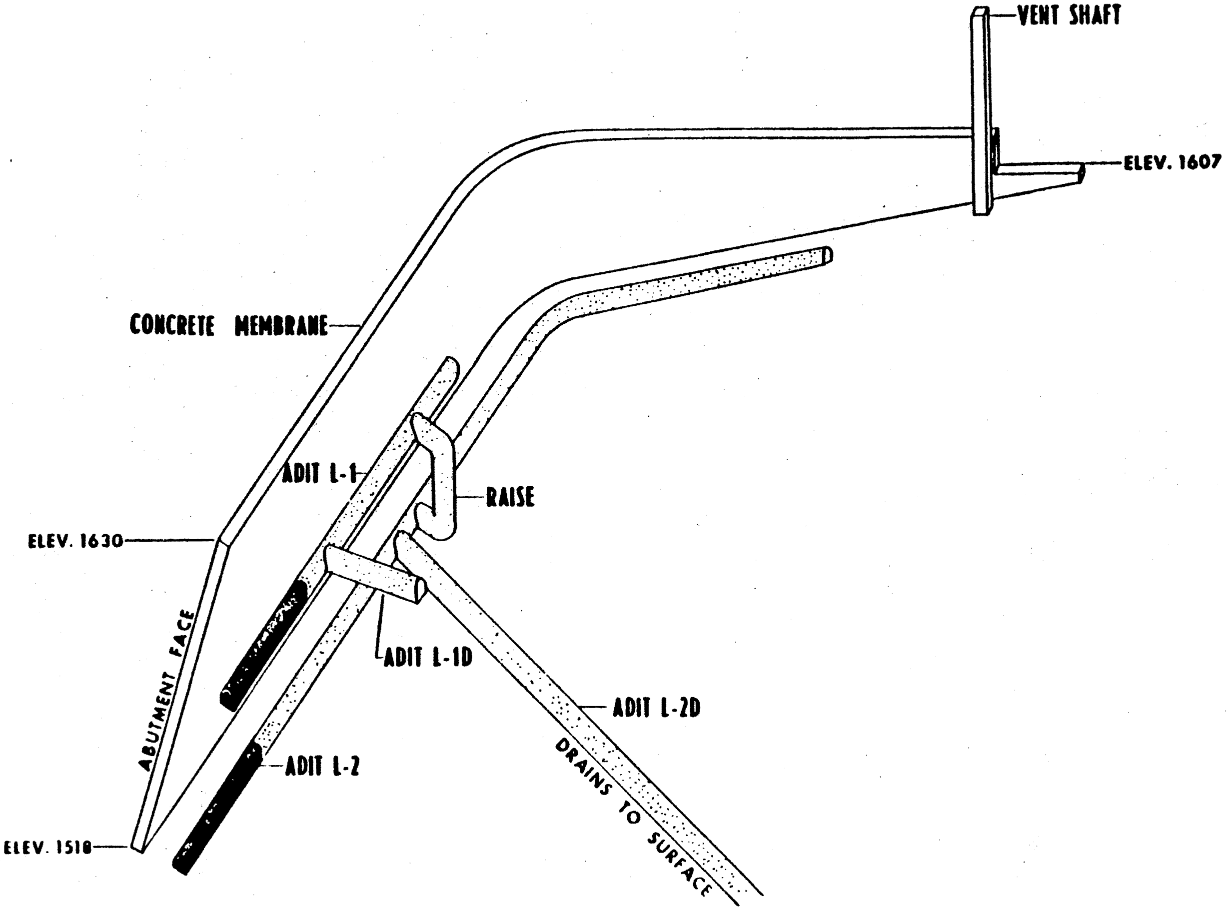
Source: (24)

Figure 107. Details of Concrete Cutoff Wall and Toe Block at R. D. Bailey Dam, West Virginia.

the trench for the cutoff wall was by presplitting and production blasting (24).

The extensive solutioning within the limestone of the left abutment for Garthright Dam in Virginia required a positive cutoff wall upstream of the adits to control leakage. A concrete cutoff wall 8 feet wide with a maximum height of 111 feet was constructed 820 feet into the left abutment as shown in Figure 108. The concrete wall is embedded three feet into an underlying formation of sound limestone. Construction was initiated by driving the sill drift, 8 feet wide and 12 feet high by the standard drill-shoot-muck-support cycle, to a 7.5 foot square vent shaft which had been simultaneously sunk to serve for ventilation, services, and emergency exit. Construction proceeded by placing concrete in the base of the drift, leaving a 6-foot high opening above each pour. The opening was enlarged to 16 feet in height for the succeeding drift, and concrete placement was repeated. Concrete placement was accomplished from an overhead trolley by retreating from the abutment portals back to the vent shaft. The concrete cutoff wall was completed in just under three years (10).

A concrete cutoff wall was constructed from the top of the existing embankment at Wolf Creek Dam in Kentucky. The concrete wall was installed to provide a positive cutoff in the cavernous limestone underlying the dam and was initiated as a result of recurring underseepage and internal erosion within the dam foundation. The adopted method for the construction of the cutoff wall was an outgrowth of several years of experience with deep foundation walls in difficult soils. However, due to the concern over the stability of the dam with high reservoir pool, the wall was constructed of primary (cased) and



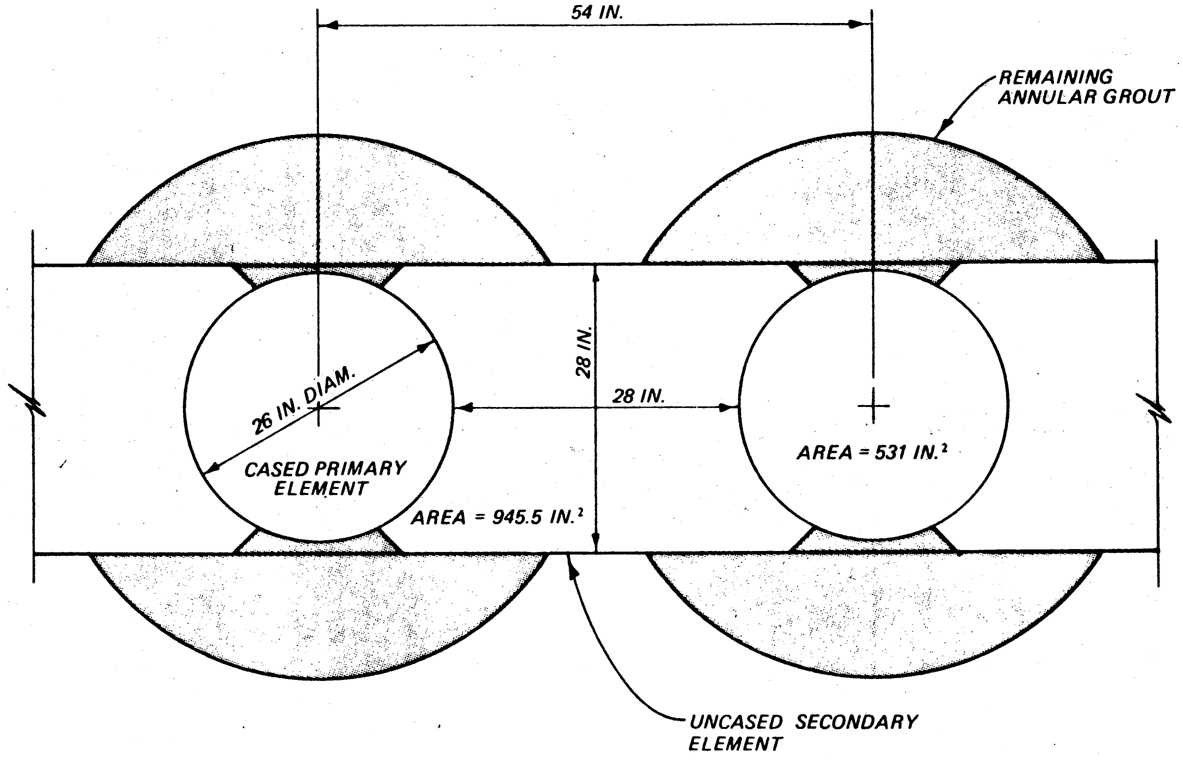
Source : (10)

Figure 108. Concrete Cutoff Wall in Left Abutment at Garthright Dam, Virginia.

secondary elements (uncased) rather than of panels which is the more conventional method for foundation projects (28). A schematic of the concrete cutoff wall is shown in Figure 109. The concrete cutoff wall is 2,240 feet long and has a maximum depth of 278 feet. The control of the vertical alignment of the primaries is critical to the success of this technique of constructing a concrete cutoff wall. Very strict limits need be imposed on the vertical alignment of the primary elements because only then is it possible to construct secondary elements which continuously follow the primaries, thereby insuring the integrity of the concrete wall (24). Both the primary and the secondary elements were cored to determine the effectiveness of the concrete wall as a positive cutoff. Voids were discovered only in the primaries, the smaller of the two elements, and thereby casts some doubt on the integrity of concrete placed by the tremie method in excavations of small areal extent (28). However, at Wolf Creek the interior steel casings for the primary elements were left in place.

During initial construction, the blasting and removal method used at Patoka Dam is very cost effective. However, this method is only cost effective for trench excavation of moderate depths, up to the 30-foot depth at Patoka Dam, because of the reach of hydraulically operated removal equipment. For deeper excavations much slower hoisting equipment would be required for rock removal. Also in narrow trench excavation in rock, blasting operations are usually conducted in 4- to 5-foot lifts (131). Therefore lateral supports and dewatering would be required for deeper excavations to maintain a safe, effective work area for blasting personnel preparing the next charge.

The method of overlapping holes used at R.D. Bailey Dam produces a



Source: (28)

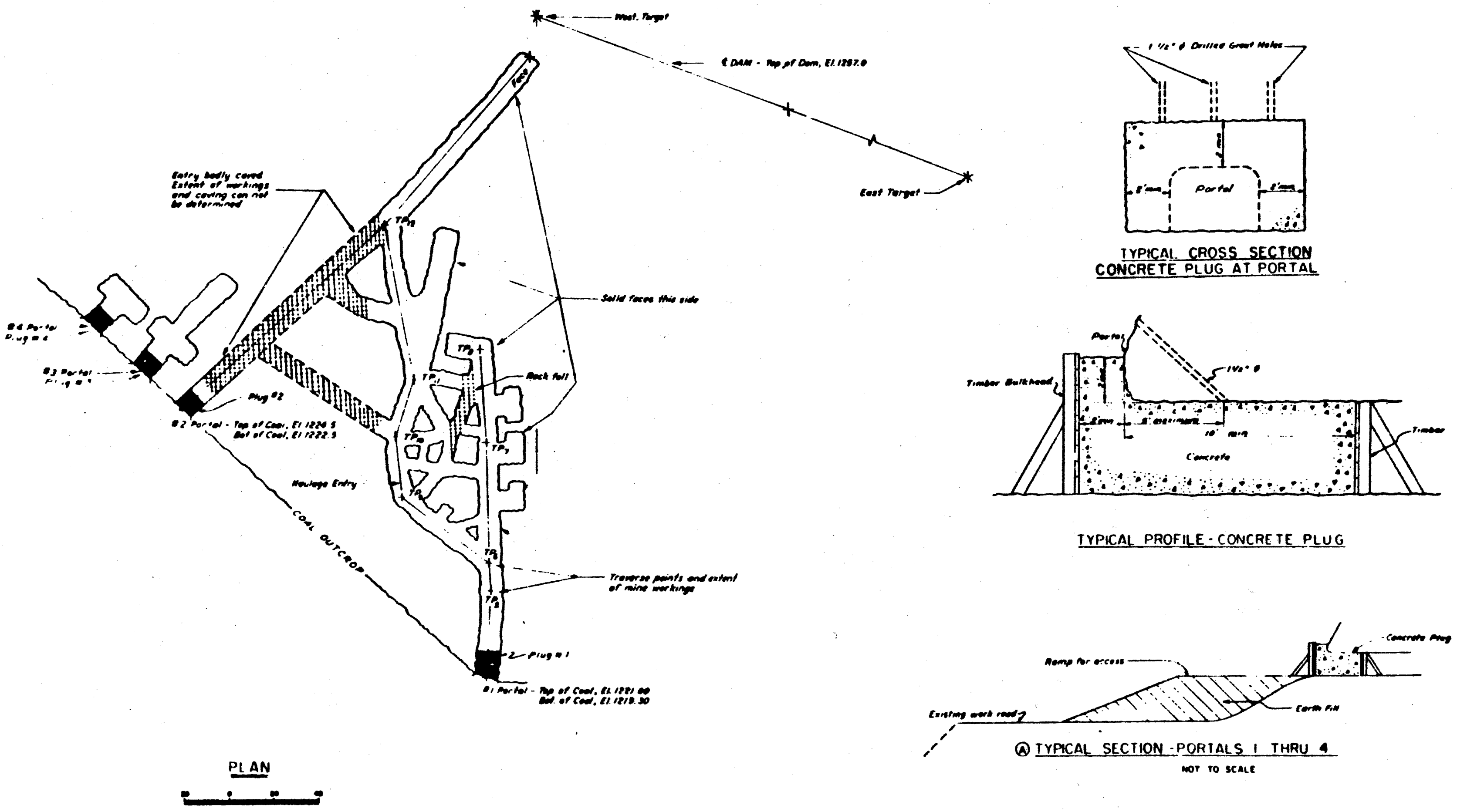
Figure 109. Schematic of Concrete Cutoff Wall at Wolf Creek Dam, Kentucky.

continuous row of overlapping concrete piers to provide a positive cutoff in rock or in pervious soil. As mentioned as treatment of pervious foundation soils, one of the deepest cutoffs of this type was constructed beneath the upstream cofferdam of the Mancouagon V Project in Quebec. That cutoff extends through alluvial sediments with boulders to a maximum depth of 250 feet (30). The two major concerns of deep excavations using overlapping holes are maintaining extreme limits for vertical alignment, and voids in tremie-placed concrete within small areas. Unlike the method used at Wolf Creek Dam, any casings for the primary holes would require removal prior to drilling the secondary holes. The near-vertical alignment required to construct a continuous concrete membrane would be less difficult to maintain in foundation rock than in foundation soils containing boulders because of deflections of the drill bit caused by hitting boulders.

The hammers used to drill the overlapping holes at R.D. Bailey Dam would be unsafe at Wolf Creek Dam. The high-pressure air surges used to blow cuttings out of the hole during excavation could possibly blow holes through the cavity fillings and endanger the dam (24).

Mine Plugging

Occasionally, abandoned mines may be located in or near the abutments at a damsite or along the reservoir rim. A more common finding is abandoned oil or gas wells in the reservoir floor. The mines and wells should be plugged to prevent contamination of reservoir water and, if near the dam, to prevent underseepage through the dam foundation. The coal mine plugging undertaken at Curwensville Dam in Pennsylvania is shown in Figure 110. At Curwensville Dam the abandoned



PLAN

TYPICAL CROSS SECTION CONCRETE PLUG AT PORTAL

TYPICAL PROFILE - CONCRETE PLUG

Ⓐ TYPICAL SECTION - PORTALS 1 THRU 4

NOT TO SCALE

Source: (100)

Figure 110. Mine Plugging at Curwensville Dam, Pennsylvania.

coal mines are located in the ridge between the spillway and the left abutment. The lowest elevation of the mines is 14 feet below the spillway crest (100). The mines at Curwensville Dam had been abandoned, but even active mines have created similar situations.

CHAPTER VI

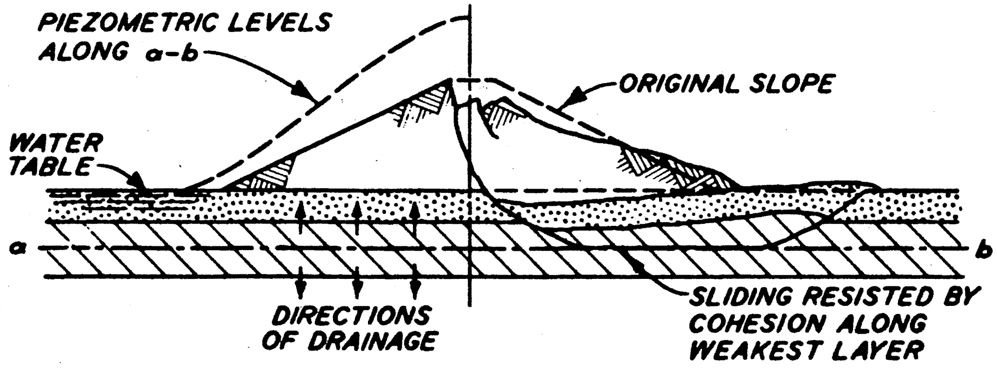
SOFT CLAY FOUNDATIONS

General Design Considerations

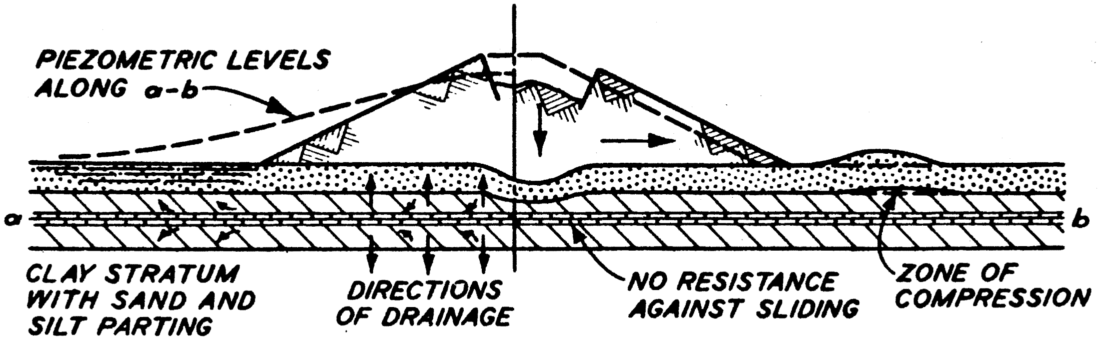
A significant development that has occurred over the past 30 years is the construction of earth dams over deep, soft clay foundations. This situation has resulted from policy changes relating to the design purpose of newer dams. Initially, within the United States, major dams were constructed in arid regions to provide water supply reservoirs for irrigation or in mountainous regions to supply hydroelectric power. The siting of these dams was more regional and allowed greater latitude in selecting favorable damsites. After flood control became a recognized purpose, the siting of the dam became tied to a specific stream or to a specific reach of a stream. As a result of this specific siting of dams, damsites are quite often less than ideal, with little or no latitude in selecting more favorable sites. Damsites that feature soft clay foundations are considerably less than ideal. Because the soft clay foundations are sufficiently impervious to preclude design features for underseepage, design considerations involve the stability of the dam.

Soft clay foundations refer to fine-grained foundation soils that are wholly or partially composed of normally consolidated clays. The clays are normally consolidated in a geological sense, but, actually, these soils exhibit the characteristics of lightly overconsolidated clays with overconsolidation ratios of from 1 to 2 (48). The slight overconsolidation is the result of slight increases in effective stress on the clays due to fluctuations of the overlying groundwater table. The condition of total saturation contributes to stability problems. As the foundation is loaded a portion of the load is transmitted to and carried by the soil particles which deform elastically or become rearranged, but with no change in volume. The remaining portion of the load is carried by stress within the contained water as pore pressure (62). The impervious nature of the clay prevents immediate or short-term dissipation of the pore pressure. Typical embankment failures on soft clay foundations are illustrated in Figure 111.

Stability analyses of embankments on soft clay foundations are appropriately conducted as total stress analyses using unconsolidated-undrained shear strengths. The obstacle to effective stress analyses is the difficulty of predicting appropriate pore pressures. A correlation between pore pressures measured in the laboratory and pore pressures that may develop in the field is difficult to achieve. Total stress analyses do not require independent pore pressure analyses. The ordinary method of slices consistently provides reliable solutions. Bishop's method produces unreasonable stress situations when applied to steeply sloping failure surfaces which are common for soft clay foundations, and the more rigorous methods develop numerical convergence problems. Evaluation of the mechanics of total stress analyses using the ordinary method of slices determined this method to be within 10 percent of results calculated from the finite element method, which is assumed to provide the true factor of safety (21). Unconsolidated-



a. HOMOGENEOUS CLAY STRATUM



b. CLAY STRATUM WITH PERVIOUS PARTING

Source: (54)

Figure 111. Typical Failures of Earth Dams on Soft Clay.

undrained shear strengths of fully saturated foundation clays obtained from laboratory testing provide the most accurate data for stability analyses, but are very sensitive to disturbance during sampling, transporting, and testing. Methods have been developed to correct the shear strengths to account for disturbance (8).

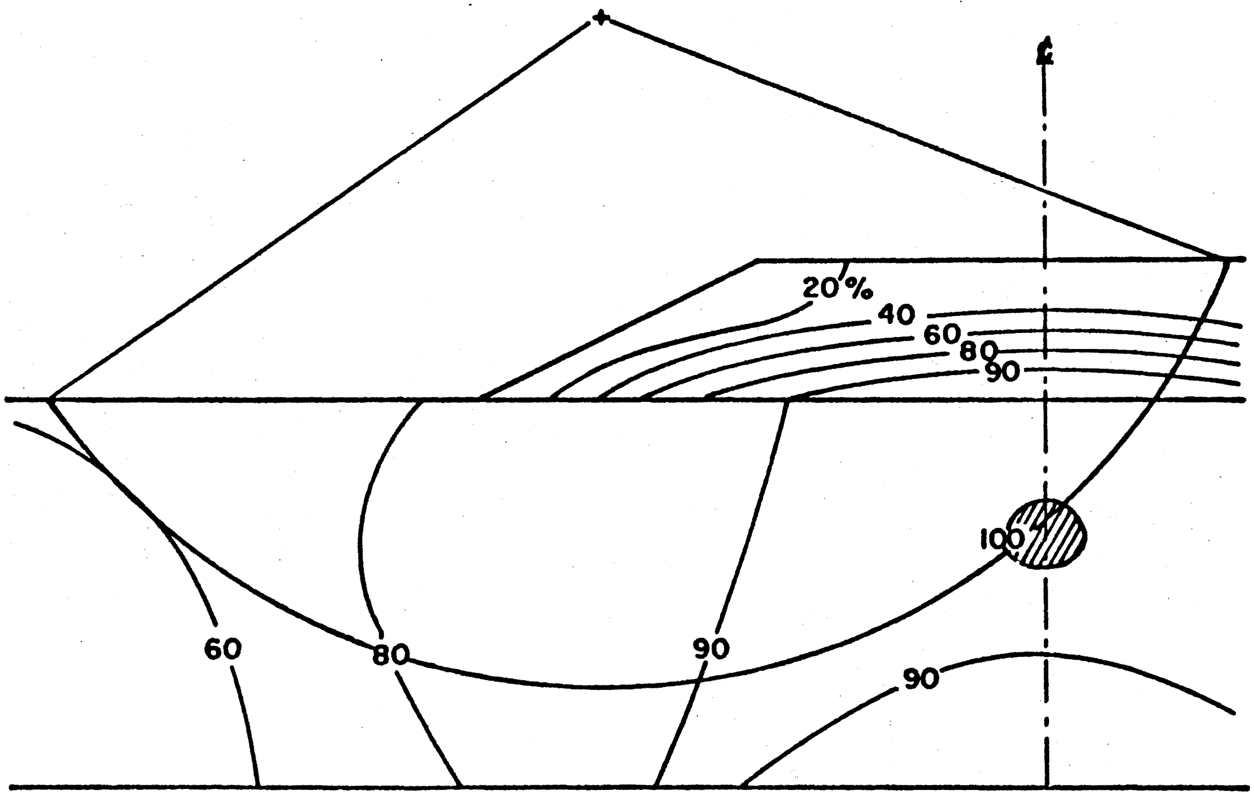
A recent survey of failures of engineering structures on soft clay foundations contained data of failures of embankments and failures of structural footings (9). The overall average factors of safety for embankment failures and for footing failures were 1.29 and 1.14, respectively. Assuming similar and equal inaccuracies in determing the foundation shear strengths for each of the two types of structures, the difference in average factors of safety may be attributed to inaccuracies in the stability analyses regarding the interaction of the structure. Stability analyses may, perhaps, attribute more shear resistance to the embankment than is justified considering the incompatibility of the stress-strain characteristics of the embankments and of the foundation.

The consideration of greatest concern involves the incompatibility of the stress-strain characteristics of the soft foundation clays relative to the stress-strain characteristics of the stiffer embankment material. Originally the concern was that the strain along the portion of the potential sliding surface which passes through the embankment might reach failure before the foundation becomes sufficiently strained to mobilize its full strength (57). Research using the finite element method to model generalized embankments over soft foundations indicates that localized failure occurs first in the foundation and initiates progressive failure (16).

This localized failure is depicted in Figure 112. The occurrence of localized failure may be attributed to the stress-strain incompatibility of the two materials. The finite element modeling has led to the development of reduction factors for the shear strengths of both the embankment and the foundation material. Corrective shear strengths are obtained by directly reducing the cohesion, c, and the tangent of the angle of internal friction, tan ϕ, by the appropriate factor from Figure 113.

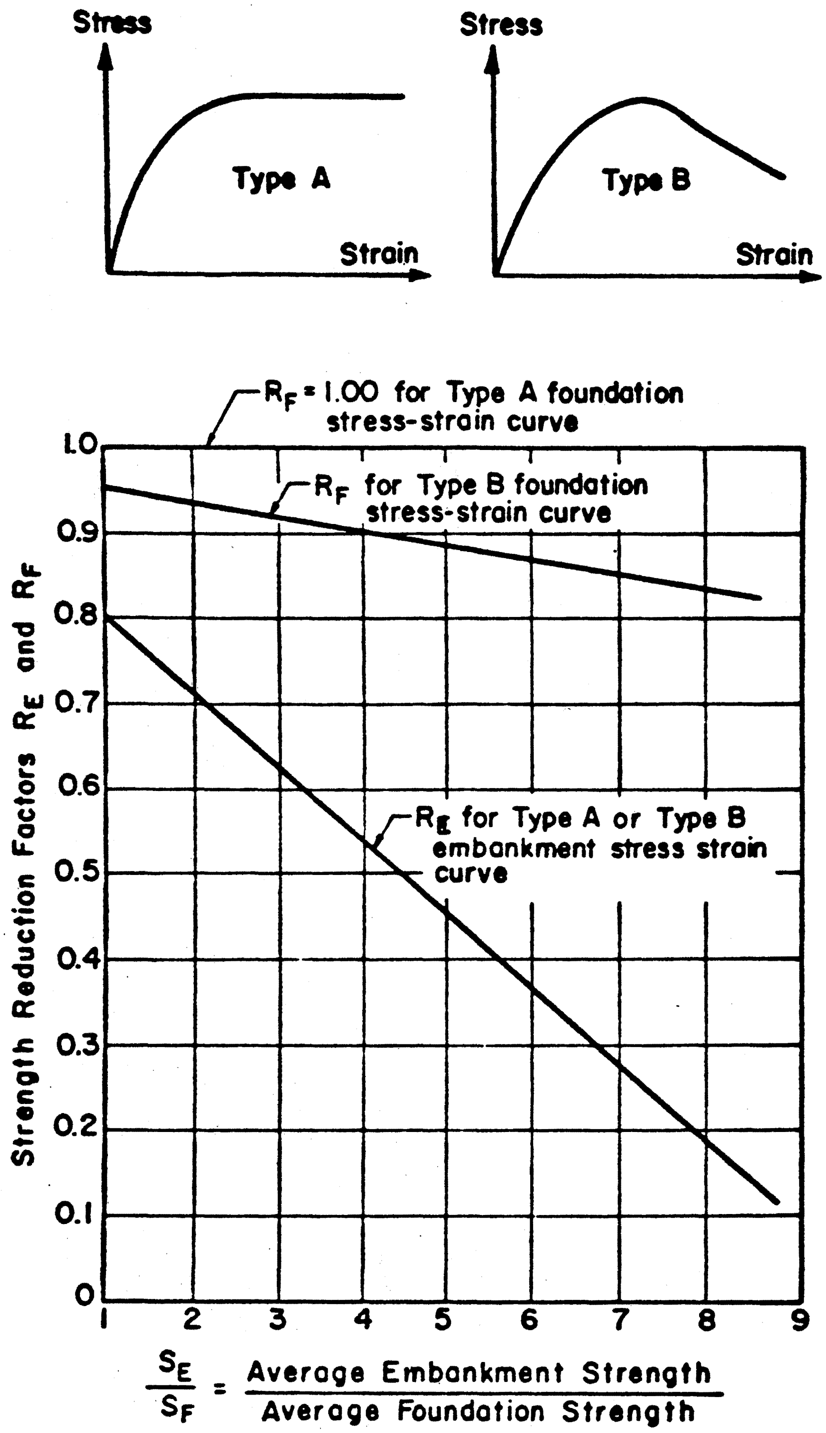
In addition to stability problems related to shear strengths, the stress-strain incompatibility may cause longitudinal cracking of the embankment which could be undetectable from the embankment surface (16). As the foundation settles and deforms outward as foundation spreading, the stiffer embankment may be dragged along on the foundation and develop tension cracks migrating from the foundation surface upward into the embankment. These cracks could adversely affect stability (62).

Pore pressures within soft clay foundations generally develop rapidly during the early stages of construction, Figure 114. If pervious strata are interspersed within the soft clay, consolidation will be accelerated during construction. If the clay is a thick, homogeneous deposit and drainage to pervious strata must follow longer paths, pore pressures can be expected to increase until the end of construction and the rate of dissipation may be quite slow. The rule-of-thumb for maximum allowable pore pressures is a piezometric head equal to the height of the fill, which gives a pore pressure ratio of about 50 percent. The pore pressure ratio is the ratio of the piezometric pressure at a point to the total stress from the fill



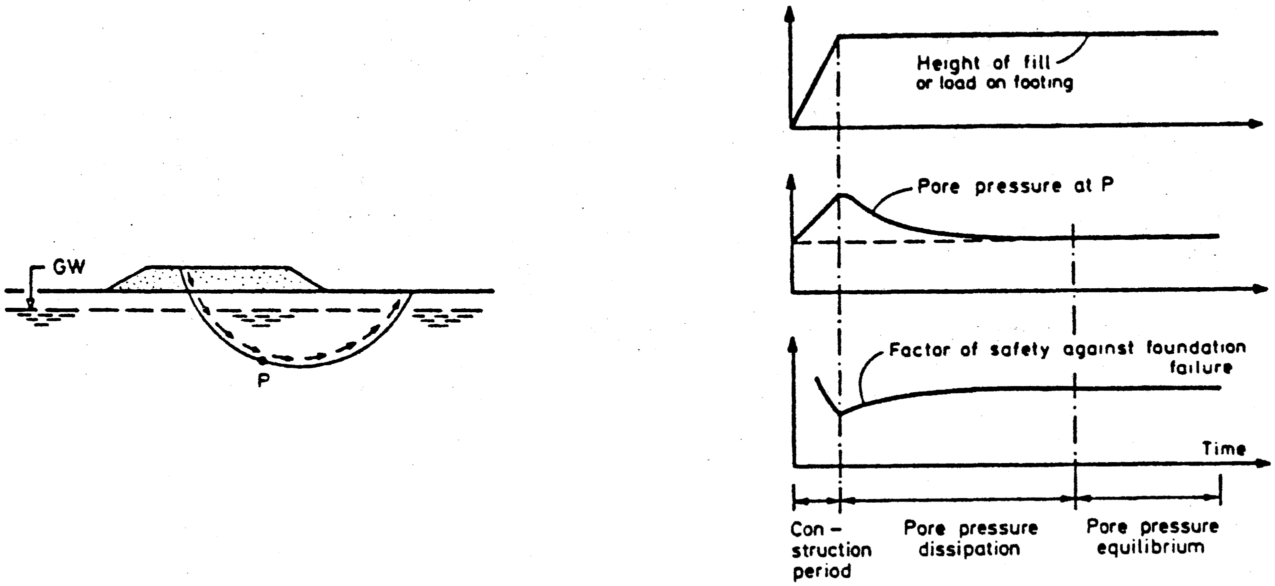
Source: (16)

Figure 112. Contours of Mobilized Strength and Critical Slip Circle at the Beginning of Local Failure.



Source: (16)

Figure 113. Correction Factors for Relating Embankment and Foundation Materials with Differing Stress-Strain Characters.



Source: (9)

Figure 114. Pore Pressures and Factors of Safety Relative to Time for Loading on Soft Clay Foundations.

height at that same point. Although earth dams have been successfully completed with pore pressure ratios near 1.0, two earth dams, Ft. Peck Dam and Waco Dam, underwent foundation failures with pore pressure ratios less than 0.7 (58).

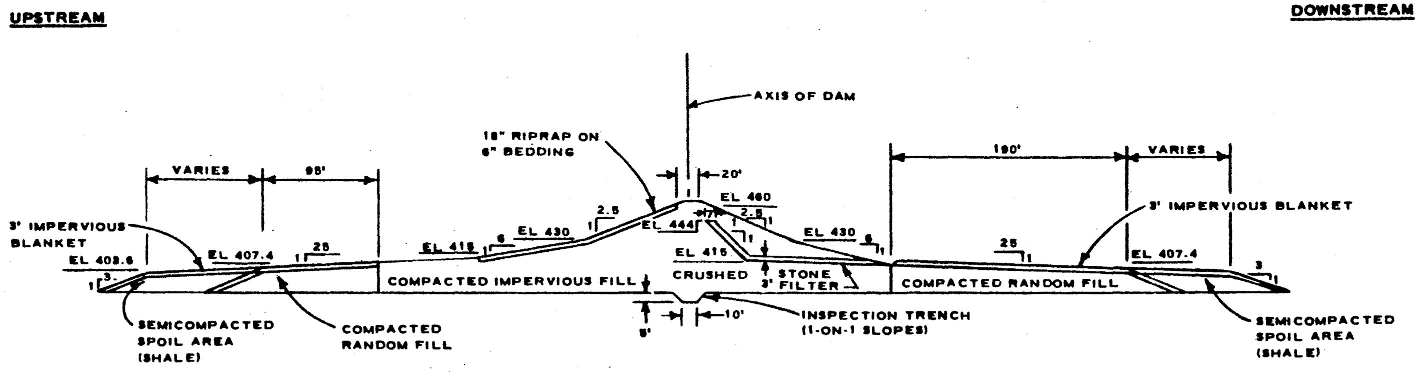
Methods to Improve Stability

Flattening Embankment Slopes

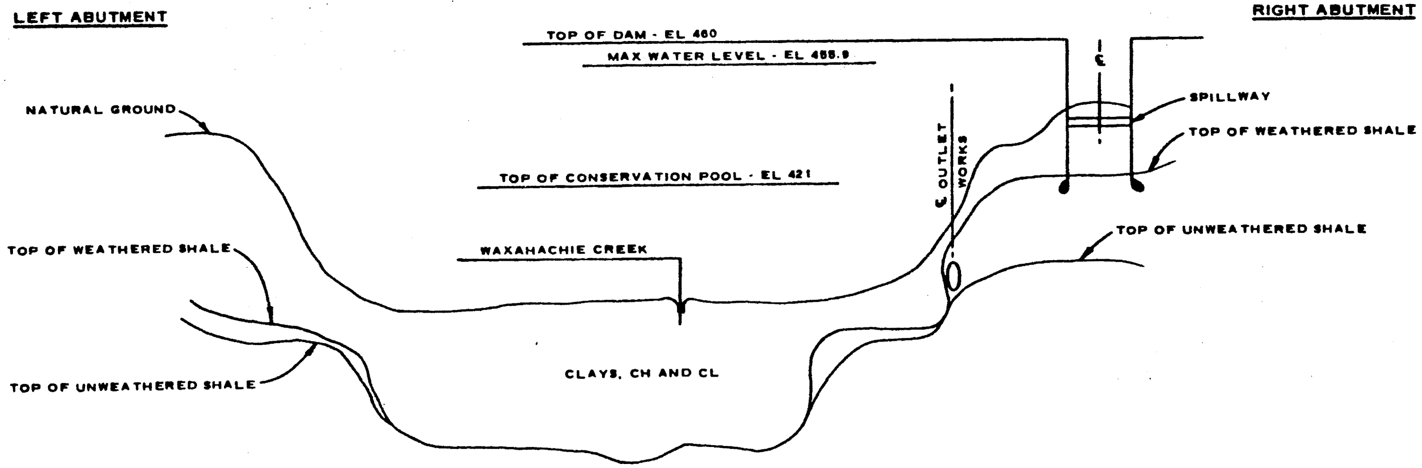
Flattening the side slopes of the embankment reduces the magnitude of the average shear stress along the potential surface of sliding. Flattening the side slopes will either directly provide additional resistance or cause a longer potential surface of sliding by forcing the failure arc deeper in the foundation. A good example of an earth-fill dam with flattened side slopes is Bardwell Dam in Texas and presented in Figure 115. The embankment of Bardwell Dam overlies 40 feet of soft clays and has side slopes as flat as 1V on 25H (137). Tuttle Creek Dam in Kansas, Figure 116, is a rockfill dam overlying up to 27 feet of soft clay and silt. At Tuttle Creek Dam, berms were provided in the closure section and over soft areas near both abutments (69).

Sand Drains

The use of sand, or other pervious material, to facilitate drainage from within a dam embankment and foundation is an integral part of the design and construction. Vertical or inclined filters intercept and convey embankment seepage from impervious cores. Horizontal filter blankets in the downstream portion of embankments



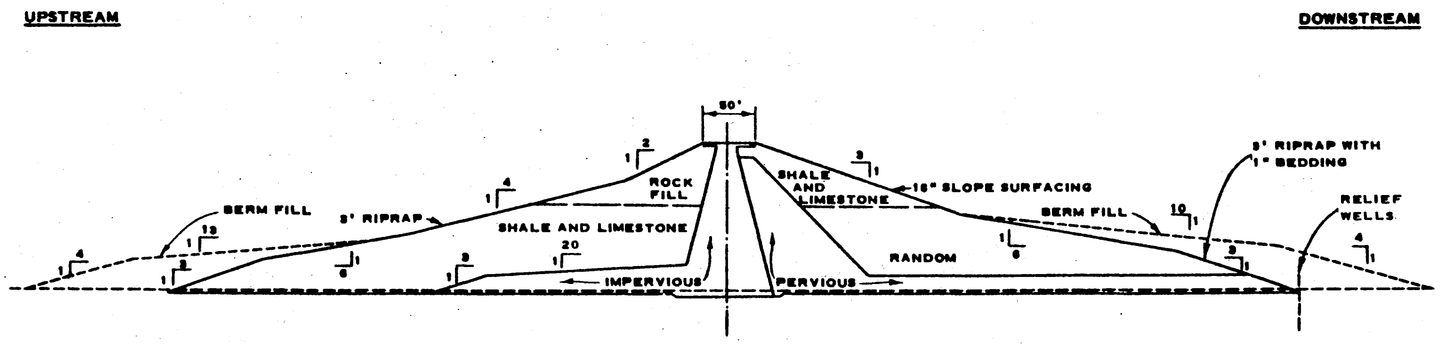
TYPICAL EMBANKMENT SECTION



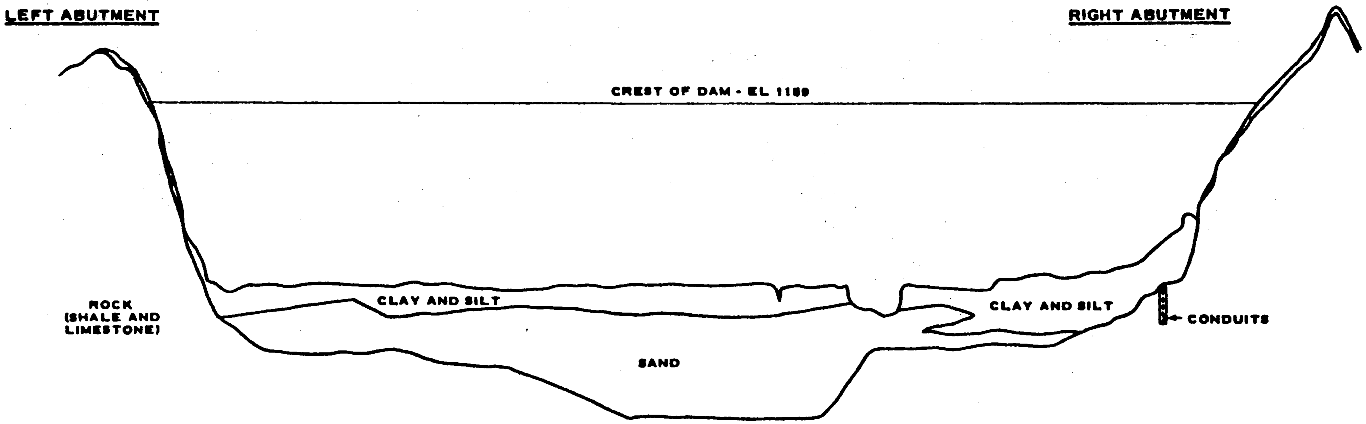
VALLEY CROSS SECTION

Source: (137)

Figure 115. Bardwell Dam, Waxahachie Creek, Texas.



TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (69)

Figure 116. Tuttle Creek Dam, Big Blue River, Kansas.

continue the conveyance of seepage to controlled outlets downstream and also intercept foundation underseepage. Both types of filters serve to maintain low piezometric surfaces through embankments.

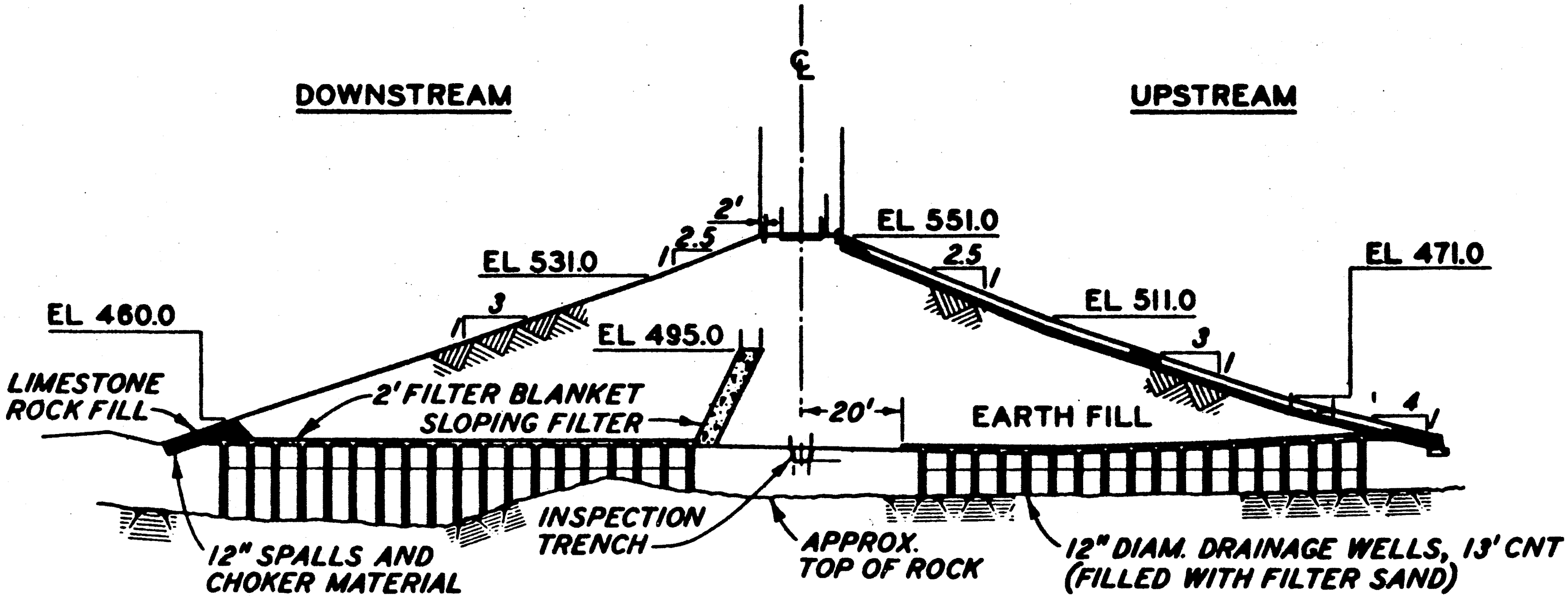
A horizontal filter blanket, or some variation, also provides an exit for excess pore water during construction thereby shortening the drainage path for consolidation of the foundation clays. Earlier techniques to provide horizontal relief at the foundation surface involved constructing a lateral system of sand- and gravel-filled trenches excavated into the foundation and perpendicular to the dam axis. At Wister Dam in Oklahoma, constructed between 1946 and 1950, trenches, 2.5 feet by 2.5 feet, were excavated on 20-foot centers and backfilled with sand and gravel. The trenches extend from about 100 feet upstream of the axis to the downstream toe and were constructed to increase the rate of consolidation and to increase the development of shear strength within the foundation (201). Currently the trend is to place a graded, horizontal filter blanket continuously over the downstream portion of the embankment foundation.

Vertical sand drains have been widely used to facilitate the drainage of excess pore water and subsequent consolidation under highway embankments. However, there is a genuine concern of dam engineers that the construction of sand drains within a dam foundation may create serious underseepage problems. Consequently, vertical sand drains have been used at very few earth dams during the initial construction. The three most prominent earth dams which utilized sand drains during initial construction are Boundary Dam in Saskatchewan, Chew Stoke Dam in England, and Rough River Dam in the United States.

Boundary Dam was constructed in 1956 and 1957 using stage construction and partially penetrating sand drains (57). Chew Stoke Dam was constructed in 1954 with sand drains to provide an increase in stability. The downstream slope of Chew Stoke Dam had a calculated factor of safety against foundation failure of 0.8 using the total stress, $\phi = 0$, analysis. The spacing of the sand drains was designed to provide a factor of safety of 1.5 using the effective stress analysis with predicted pore pressures. Field observations of pore pressures indicated that the actual factor of safety was somewhat greater than 1.5 because of greater horizontal permeability of the clay due to stratification, and unaccounted for in the effective stress analysis (7).

Rough River Dam, constructed by the Corps of Engineers and completed in 1958, provides a good example of the value of vertical sand drains to accelerate consolidation and strength gain of soft clay foundations. The soft clay foundation on the left bank at Rough River Dam has natural drainage layers which provided relief of excess pore water during construction. The soft clay foundation on the right bank has a maximum depth of 40 feet and has no natural drainage layer. On the basis of unconsolidated-undrained tests, adequate stability could not be achieved for the end-of-construction condition because of the low shear strengths of the foundation clays. Sand drains, 12 inches in diameter, were constructed through the full depth of the right bank foundation to provide short horizontal drainage paths for rapid consolidation, thereby permitting higher shear strengths in stability analyses for the end-of-construction condition (71). A typical section of Rough River Dam for the right bank is shown in Figure 117.

Six settlement gages and eight piezometers were installed in the



Source: (71)

Figure 117. Sand Drains at Rough River Dam, Kentucky.

foundation of Rough River Dam to monitor the rate of settlement and the development of pore pressures within the embankment during construction. Three settlement gages were placed on the right bank and three on the left. The settlement data obtained from these gages are listed in Table XV. All eight piezometers were placed in the right bank foundation to monitor the effectiveness of the sand drains.

TABLE XV

SETTLEMENT AT ROUGH RIVER DAM

Gage	Settlement, ft.	Settlement Rate, %
Left Bank		
1	1.29	93
2	2.06	97
3	1.33	98
Right Bank (Sand Drains)		
4	0.95	98
5	1.83	97
6	0.73	100

Source: (71)

Stage Construction

Embankments may be constructed in increments or stages, on soft clay foundations that have some degree of natural drainage. Each stage is constructed to the extent allowed by the respective shear strength of the foundation. Subsequent pore pressure dissipation and corresponding strength gain during scheduled layover periods allows construction of succeeding stages. Layover periods may result in considerable idle time for the construction contractor, or in substantial mobilization and demobilization expenses if construction is separated into contracts relative to the different stages. Therefore, proper engineering design requires comparative cost analyses of an embankment designed with flattened side slopes or with stability berms which compensate for the low shear strength of the soft clay foundation. Table XVI presents a sampling of earth dams constructed by stage construction.

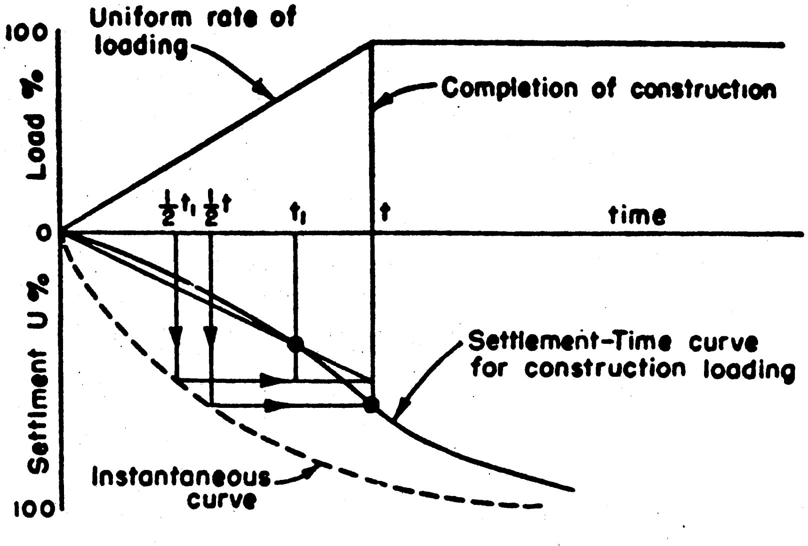
A consolidation curve based on a rate of load application, rather than on an instantaneous loading, serves to estimate the percent consolidation of specific construction times. A common method of adjusting an instantaneous curve to establish a consolidation-time curve for construction is based on the assumption that the settlement at the end of the construction period is equal to the settlement at half-time for instantaneous loading (62). The plotting of the consolidation-time curve for construction loading is illustrated in Figure 118. For stage construction a separate consolidation-time curve is plotted for each increment of fill, with the initial time, t = 0, of each separate consolidation-time curve corresponding to the starting

TABLE XVI

EARTH DAMS CONSTRUCTED BY STAGE CONSTRUCTION

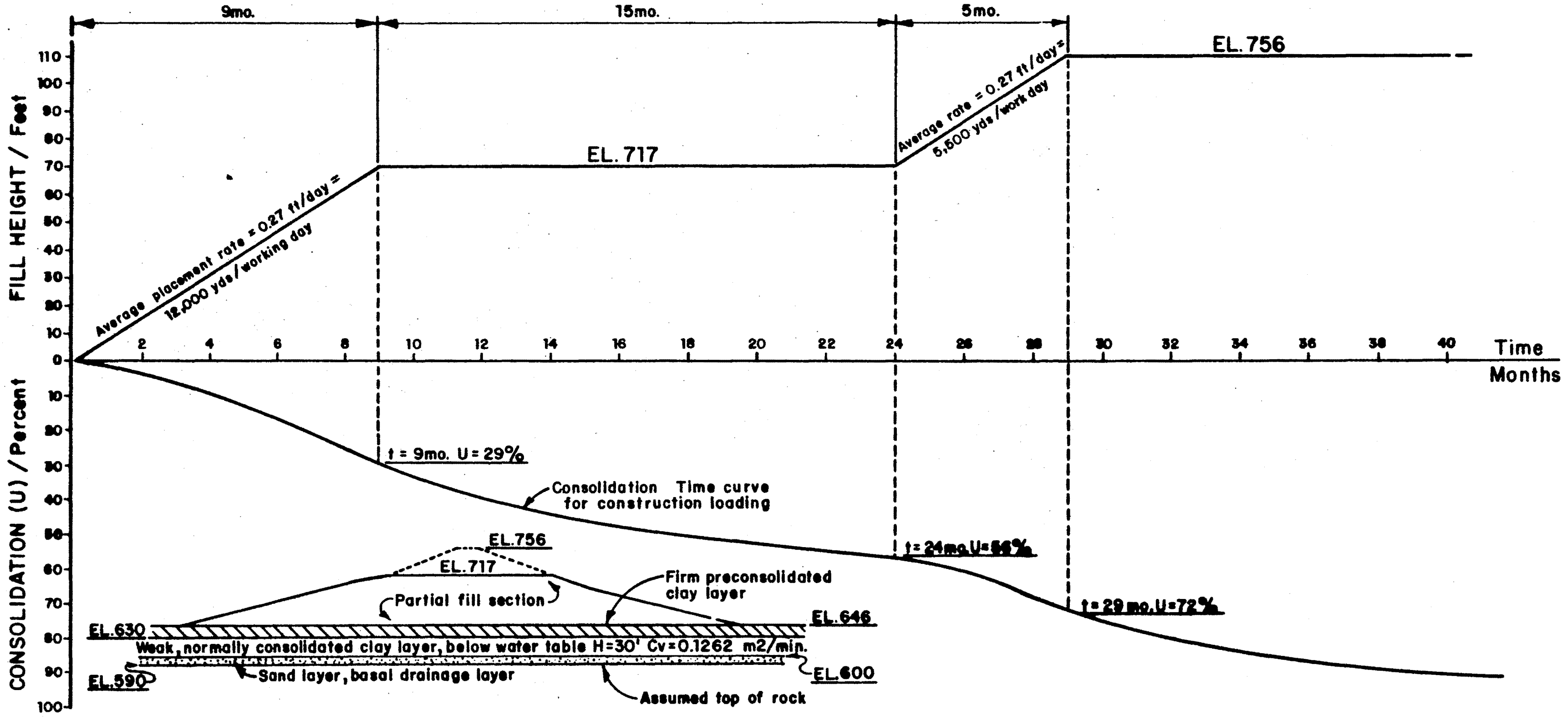
NAME OF DAM	YEAR	SIDE SLOPES	F.S. w/o	CLAY LAYER	GROUND SURFACE	STAGE 1			STAGE 2	CONSOLIDATION	F.S. FINAL
						TOP	CONSTRUCTION	LAYOVER	TOP		
Tom Jenkins, Ohio	1949	1V-on-3.5H	1.00	12-20 feet	el. 690	el. 730	65 days	200 days	el. 765	35 %	1.21
Curwensville, Pennsylvania	1965	1V-on-3.5H	-	25-36 feet	el. 1175	Controlled rate of placement over three seasons				-	-
Conestoga, Nebraska	1964	1V-on-3H	0.85	40 feet	el. 1215	el. 1240.5	160 days	60 days	el. 1260	-	1.80
Boundary, Saskatchewan	1957	1V-on-3H	-	35 feet	el. 1775	el. 1805 el. 1825	35 days 10 days	180 days 25 days	el. 1850	-	-
Willard, Utah	1964	1V-on-3H	-	several hundred	el. 4199	el. 4221	1156 days	700 days	el. 4235	-	-
Birch, Oklahoma	1976	1V-on-3H	1.00	35 feet	el. 720	el. 770	60 days	90 days	el. 795	50 %	1.30
Skiatook, Oklahoma	1984	1V-on-3.5H	0.90	45 feet	el. 645	el. 717	450 days	1140 days	el. 756	70 %	1.28

time of the respective fill increment. The ordinates of each construction-time curve are added to obtain ordinates for the overall consolidation-time curve for the final embankment height. The percent consolidation must correspond to the loading of the final embankment height in order to estimate increases in shear strengths to resist the final loading. The percent consolidation of the foundation clay relative to loading of the final embankment will provide a design estimate of the increase in shear strength due to consolidation. The consolidation-time curve for Skiatook Dam is presented in Figure 119.



Source: (206)

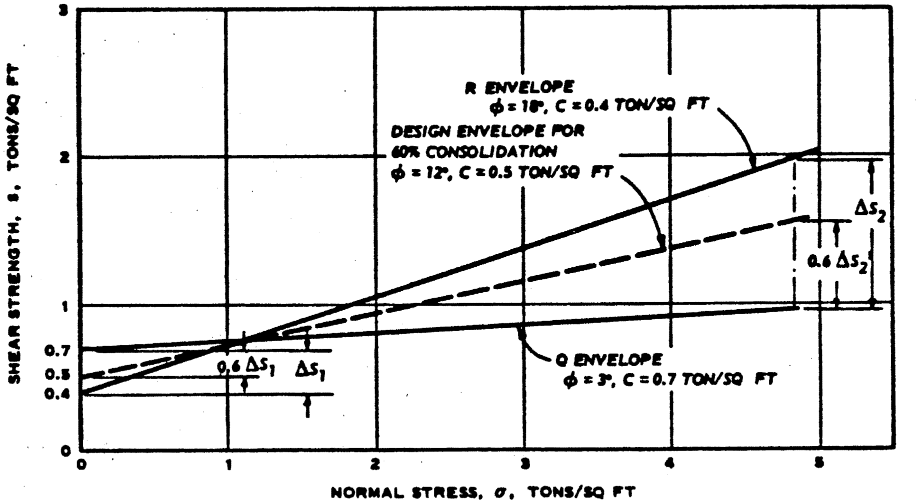
Figure 118. General Consolidation-Time Curve for Construction Loading.



Source: (124)

Figure 119. Consolidation-Time Curve of Construction Loading for Skiatook Dam, Oklahoma.

During design the initial unconsolidated-undrained shear strengths can be used to establish the height of the first stage. The stability of subsequent stages and the final embankment can be analyzed using effective stress analyses with estimated pore pressures. Or, the shear strength can be estimated by interpolating between the consolidated-undrained strength envelope and the unconsolidated-undrained strength envelope for use in a total stress analysis, thereby eliminating the need to estimate pore pressures (130). This method of estimating the shear strength by interpolating between the strength envelopes is illustrated in Figure 120.



Source: (130)

Figure 120. Interpolation of Strength Envelope for Partial Consolidation.

The traditional method to verify the consolidation and subsequent strength gain of the foundation clays is to monitor settlement and pore pressures during construction. Effective stress analyses using consolidated-undrained strength parameters with construction pore pressures provide satisfactory incremental factors of safety and the final factor of safety for end of construction.

For stage construction of Birch Dam, the height of the first stage was established using unconsolidated-undrained strengths of the foundation clay in a total stress analysis which assumed no increase in strength of the foundation during construction of the first stage. The layover period required for about 95 percent consolidation was estimated from a consolidation-time curve. At the end of the layover period during construction, undisturbed samples of the foundation were obtained through the first-stage embankment. Unconsolidated-undrained shear strengths from these undisturbed foundation samples were used to analyze the stability of the final embankment. At that time the shear strength of the foundation clay had increased sufficiently to allow completion of the embankment to full height (140). A similar approach was used for stage construction of Conestoga Dam (116).

The design and construction of Birch Dam and Conestoga Dam was a very conservative method for stage construction and did not take into account the consolidation and subsequent strength gain that occurs during fill placement. However, for Skiatook Dam, the increase in shear strength of the soft foundation clays due to consolidation during concurrent fill placement was considered in stability analyses in order to restrict construction to only two stages. The increase in shear strengths due to consolidation during concurrent fill placement was required to achieve the minimum factor of safety of 1.3 as required for end of construction by the Corps of Engineers. To further insure against instability of the embankment, a companion stability analysis based on the method used at Birch was required to achieve a minimum factor of safety of 1.0 (124).

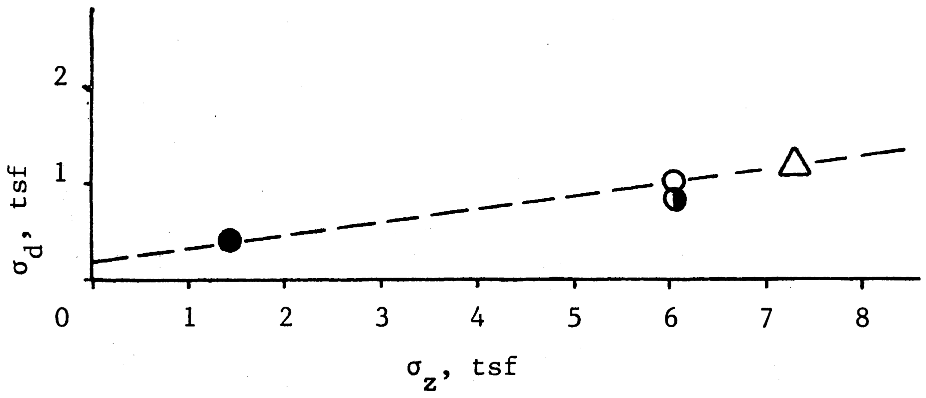
In addition to monitoring settlement and pore pressures during construction to verify the stability of Skiatook Dam, a new analytical method was developed and implemented to verify the increase in shear strength of the soft clay foundation. In simplest terms, this new method undertook conducting a full-scale consolidated-undrained shear test. Undisturbed samples of the foundation clays were obtained during four distinct periods: (1) the initial design phase prior to construction; (2) after completion of the first stage; (3) after the required layover period following completion of the first stage; and (4) a substantial time after completion of the embankment. Unconsolidated-undrained shear strengths of these samples were obtained from triaxial tests. Since the samples were totally saturated, the only component of their shear strength was apparent cohesion. The average value of cohesion from each of the sampling periods was plotted as the shear strength at failure to a corresponding value of consolidation or confining pressure. The consolidation pressure was taken as the overburden pressure for the first sampling period, the overburden pressure plus first-stage embankment load for the second and third sampling periods, and the overburden pressure plus full embankment load for the final sampling period. Consolidation had not progressed sufficiently for the second sampling period, immediately after completion of the first stage embankment. This situation is readily apparent on the plot of the full-scale test. The embankment loads were corrected for loads of finite extent in order to more accurately estimate the confining pressure (50). The shear strength envelope obtained from the full-scale consolidated-undrained test is presented in Figure 121. The predicted increase in shear strength of the soft clay foundation was verified by this method.

Electroosmosis

Electroosmosis stimulates drainage of saturated soils thereby promoting consolidation and subsequent increases in the shear strength. Electroosomosis is particularly advantageous for silty soils in which small changes in water content may produce large changes in mechanical resistance properties (14). Drainage of soils by electroosmosis is achieved by installing two lines of electrodes, one line of anodes (+) and one line of cathodes (-), and creating a direct-current electrical potential between the two lines. The presence of positive ions in the water causes flow toward the line of cathodes where the water can be removed by suction wellpoints or deep wells (13).

The only major application of electroosmosis at an earth dam within the United States was at West Branch Dam in Ohio, completed by the Corps of Engineers in 1966. Electroosmosis was used to stabilize soft clays and silts within the dam foundation through stimulated drainage and consolidation of those foundation soils (207).

The dam foundation consisted of up to 105 feet of silt grading to lean clay grading back to silt and silty sand above the top of rock, as shown in Figure 122. The foundation soils were fully saturated and unconsolidated-undrained shear strengths of portions of the lean clay



Aug 77 Construction began

May 79 Sampling 6 months after completion of first stage.

Feb 82 Sampling 38 months after completion of first stage.

Nov 82 Sampling 6 months after completion of embankment.

● Aug 77 c = 0.4 tsf
◐ May 79 c = 0.8 tsf
○ Feb 82 c = 1.0 tsf
△ Nov 82 c = 1.2 tsf

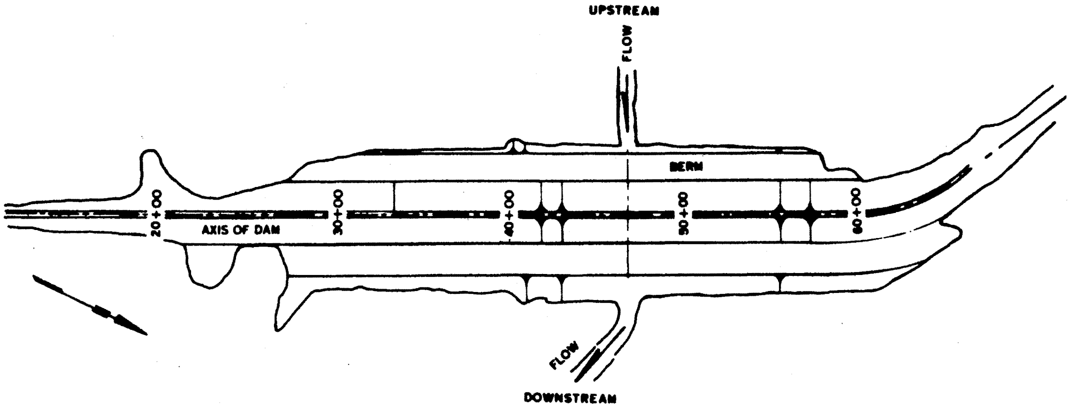
γ = 130 pcf

El. 615 approximate center of clay layer.

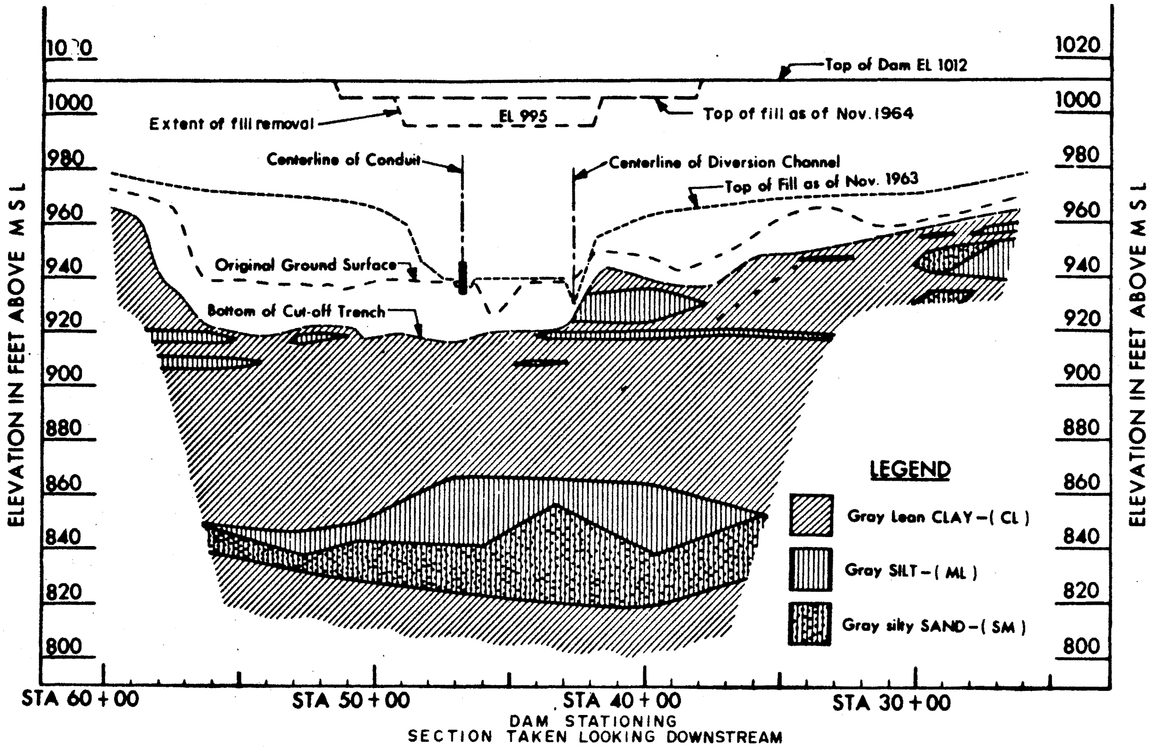
CONDITION	H FT.	γ'H TSF	γH TSF	$\sigma_z/\gamma H$	$(\sigma_z)_{emb}$ TSF	σ_z TSF	
Top of ground	30	1.4	-	-	-	1.4	El. 645
First stage	72	1.4	4.7	0.98	4.6	6.0	El. 717
Completion	111	1.4	7.2	0.82	5.9	7.3	El. 756

Source: (124)(196)(197)(198)

Figure 121. Full-Scale Consolidated-Undrained Strength Test at Skiatook Dam, Oklahoma.



(a) Embankment Plan.



(b) Valley Cross Section.

Source: (207)

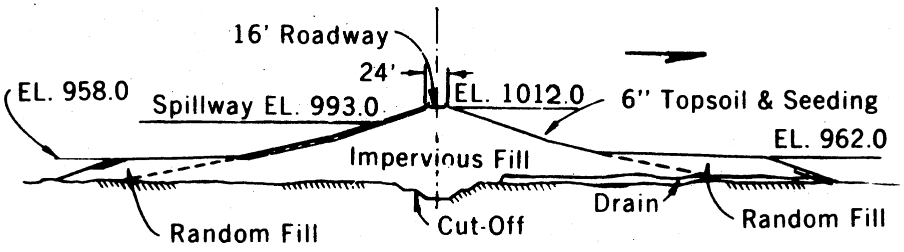
Figure 122. West Branch Dam, West Branch, Mahoning River, Ohio.

were found during testing for design to be as low as $\phi = 0$ and $c = 0.20$ tsf, with a corresponding factor of safety of 0.72 for completion.

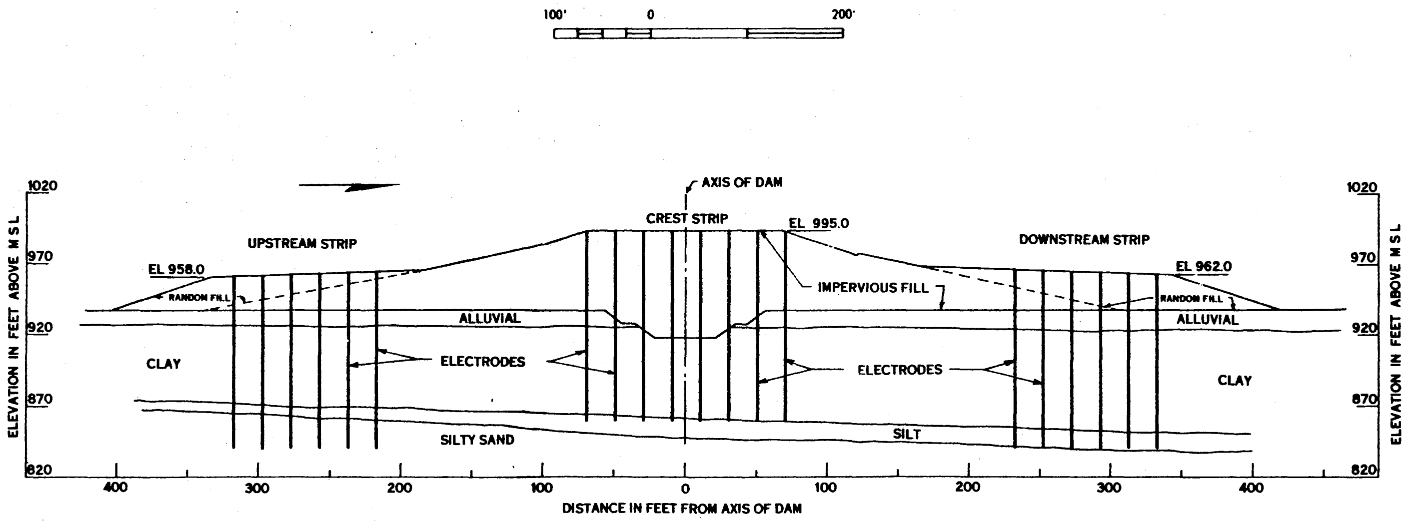
The typical embankment section for closure at West Branch Dam is shown in Figure 123. The closure section embankment was constructed from an average foundation elevation corresponding to elevation 938 to elevation 1007 (4 feet below top of dam) in 9 weeks. Longitudinal cracks were discovered near the top of the closure section, and horizontal separation of the conduit and settlement of the embankment appeared to be occurring at an accelerated rate. Analysis indicated that the embankment was in the initial stage of failure and the closure section was lowered to elevation 995 by removal of embankment material in order to unload the foundation.

Electroosmosis was provided through three treatment strips each 1,000 feet long extending through the closure section, as shown on Figure 125. Each strip was composed of electrodes on 20-foot centers in lines 20 feet apart with an alternating sequence of anodes and cathodes, both longitudinal and transverse. Eight rows of electrodes were installed along the crest and six rows each were installed along the upstream and downstream berms. The system included a total of 663 anodes and 322 cathodes. The anodes were of 2.5-inch steel pipes, except in the crest strip near the conduit where 60-pound steel railroad rails were used. The cathodes consisted of 2-inch steel pipe electrodes and eductor wells surrounded by 14-inch diameter columns of select sand. The electrodes are illustrated in Figure 124.

Electroosmosis treatment was provided for 10 months and achieved the anticipated results of appreciably lowering the piezometric level within the foundation and stabilizing the embankment. A typical



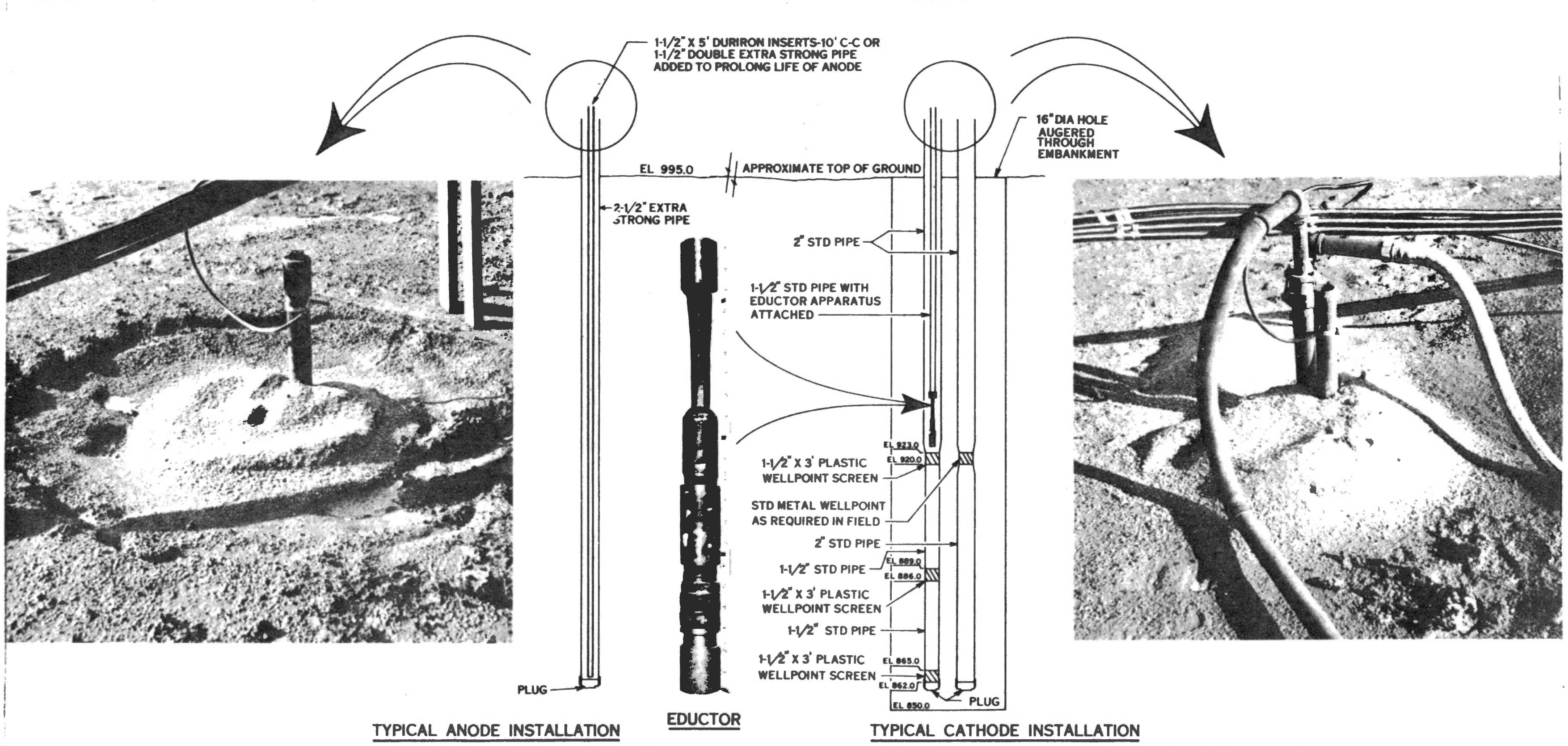
(a) Typical Embankment Section.



(b) Sectional Location of Electrodes.

Source: (207)

Figure 123. Closure Embankment of West Branch Dam.



Source: (207)

Figure 124. Electrodes, West Branch Dam.

piezometric plot and a typical settlement plot from the closure section are shown in Figure 125. The increase in shear strength due to the electroosmosis provided a minimum of $\phi = 0$ and $c = 0.50$ tsf for a corresponding factor of safety of 1.16 (207).

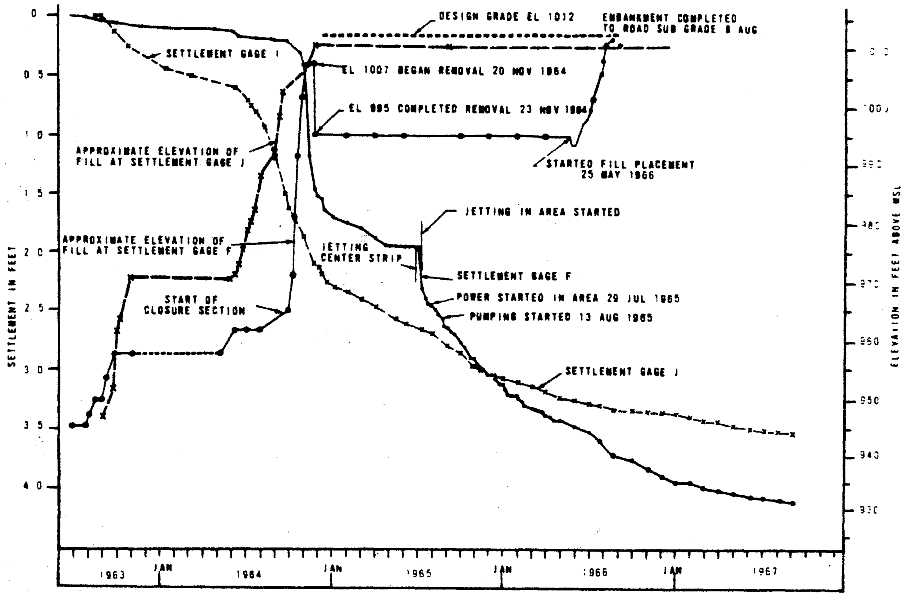
Special Design Considerations

Explorations and Testing

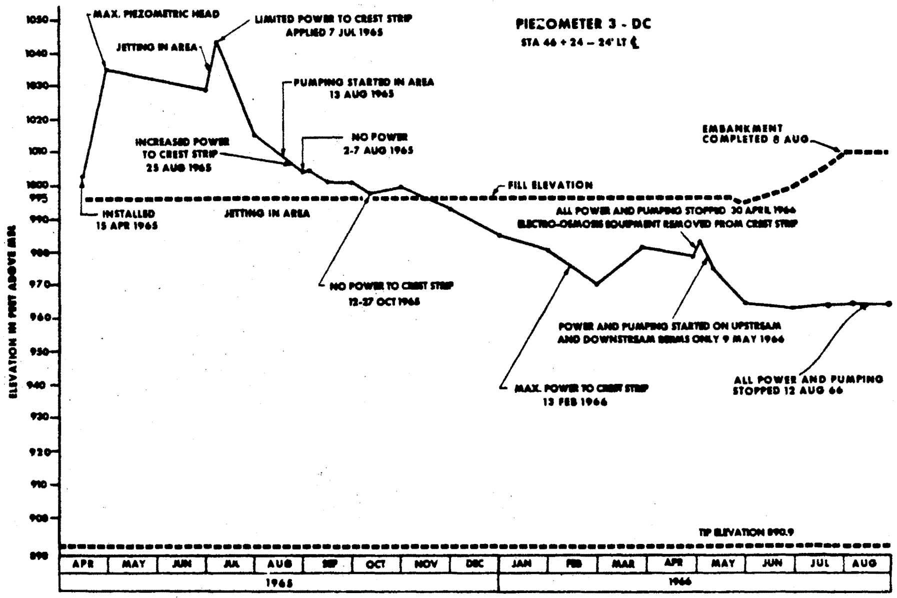
The explorations program should provide adequate investigation of the foundation to determine the existance of problem soils such as soft clays and to reasonably determine their extent. At West Branch Dam only two borings penetrated through the full depths of the clay layer, and at Skiatook Dam only one hole extended to the soft clay during initial design. Laboratory tests should involve sufficient samples to determine an adequate profile of shear strengths and consolidation data.

Stability Analyses

Stability analyses are critical for the design of construction stages, stability berms, and embankment slopes for earth dams on soft clay foundations. The type of stability analysis must correspond to the expected mode of failure. Circular arc methods, such as the ordinary method of slices, are appropriate for deep, fairly homogeneous clay foundations; however, for highly stratified foundations, wedge methods are more compatible with sliding within the weakest foundation layers. The lowest value of shear strength should be selected as representative of the foundation, or of specific foundation zones.



(a) Settlement Data.



(b) Piezometric Data.

Source: (207)

Figure 125. Typical Instrumentation Data, West Branch Dam.

And, the shear strengths of the embankment materials should be corrected to compensate for the incompatibility of stress-strain characteristics.

Instrumentation

Because of the critical nature of soft clay foundations, the installation and monitoring of instrumentation is very important. Instrumentation must be installed prior to initiating construction of the embankment in order to obtain base references for future instrumentation data. Pneumatic pore pressure transducers may be installed completely within the foundation and eliminate the construction nuisance of riser pipes. Inclinometers provide the best instrumentation within soft clays, and provide horizontal deflection data, settlement data, strain data, and, to a limited extent, piezometric data.

Removal of soft foundation clay is economical for shallow deposits and eliminates the problem. For deeper foundations where removal is not a viable alternative, a number of solutions have been successfully implemented and range from flattening the side slopes of the embankment to electroosmosis to facilitate consolidation.

CHAPTER VII

SPECIAL DESIGN PROBLEMS

Dispersive Clay

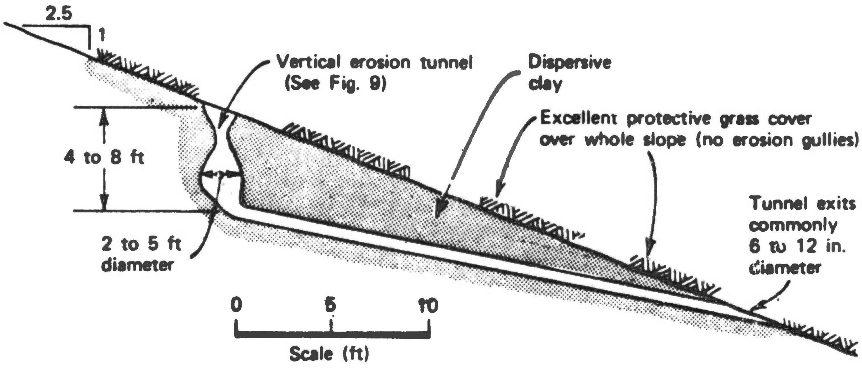
Physico-Chemical Aspects

Dispersive clays erode by a process of "dispersion" or "deflocculation" when the repulsive forces (electrical surface forces) between individual clay particles exceed the attractive (van der Waals) forces. Sodium cations within the dispersive clay structure act to increase the thickness of the diffused double water layer surrounding the individual clay particles thereby decreasing the attractive force between the particles. In the presence of water, individual clay particles become progressively detached from the clay mass and go into suspension. If the water is flowing, as seepage through an earthfill, the dispersed clay particles are then carried away (56).

Nature of Erosion

For small earth dams, typically impounding farm ponds, constructed of dispersive clays, the erosion of the clays through dispersion in seepage flow frequently causes total failure through piping of the embankment (56)(221). These dams generally impound water obtained through storm runoff as surface flow. The impounded water is relatively free of dissolved salts and therefore is able to take an appreciable amount of sodium cations and clay colloids into suspension. For major earth dams constructed of dispersive clay and impounding large reservoirs, the problems associated with dispersive clays are limited to the near-surface areas of the embankments. In areas of dispersive clays, reservoir inflow is saturated with sodium cations from groundwater leachate and cannot take into solution additional sodium during seepage through the embankment. However, storm runoff, devoid of dissolved salts, is readily capable of dissolving dispersive clay as it percolates immediately under the embankment slope surfaces. No major earth dams constructed of dispersive clay have failed or developed seepage problems as a direct result of the erosion potential inherent in dispersive clay (57).

The physical nature of the internal erosion of embankment slopes of dispersive clay is illustrated in Figure 126. The erosion develops within the embankment as piping from near the toe of the slope due to percolation of storm runoff. The eroded materials form small alluvial fans beyond the exits of the pipes, as shown in Figure 127 at Wister Dam in Oklahoma near the end of a rainstorm. The pipes extend upslope to vertical erosion tunnels which extend to the slope surface. A vertical erosion tunnel at Wister Dam is shown from the surface in Figure 128 and the accompanying pipe is exposed in Figure 129. The magnitude of the vertical erosion tunnels is well illustrated in Figure 130 which shows their presence along the embankment access road of Sardis Dam in Oklahoma. This particular portion of the slope was covered with rock spalls as protection against surface gulleying. An adjacent portion of the slope not covered by spalls is shown in Figure 131.

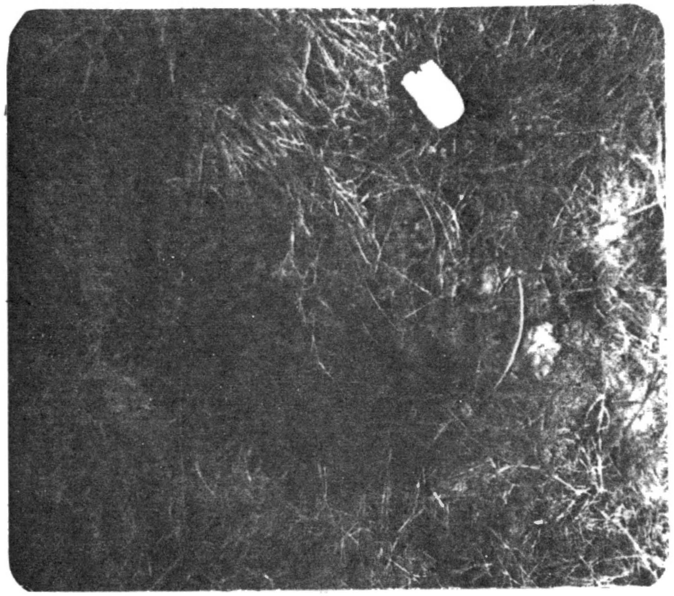


Source: (55)

Figure 126. Details of Erosion of Embankment Slopes of Dispersive Clay.

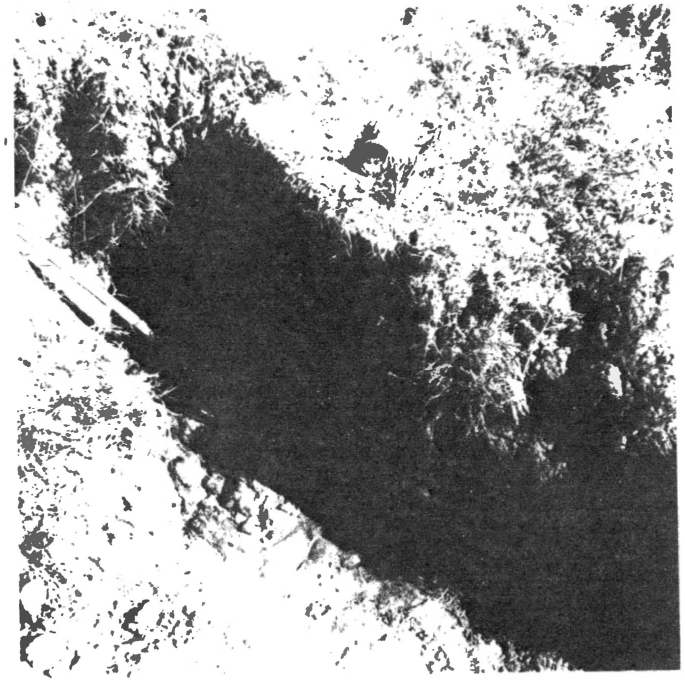
Source: (194)

Figure 127. Small Alluvial Fans Beyond Pipe Exits in Dispersive Clay, Wister Dam, Oklahoma.



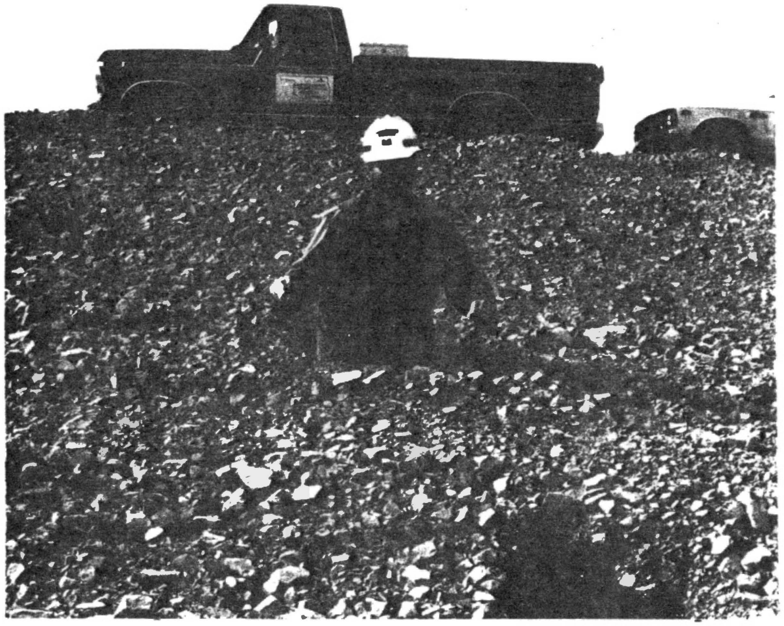
Source: (194)

Figure 128. Top of Vertical Erosion Tunnel in Dispersive Clay, Wister Dam.



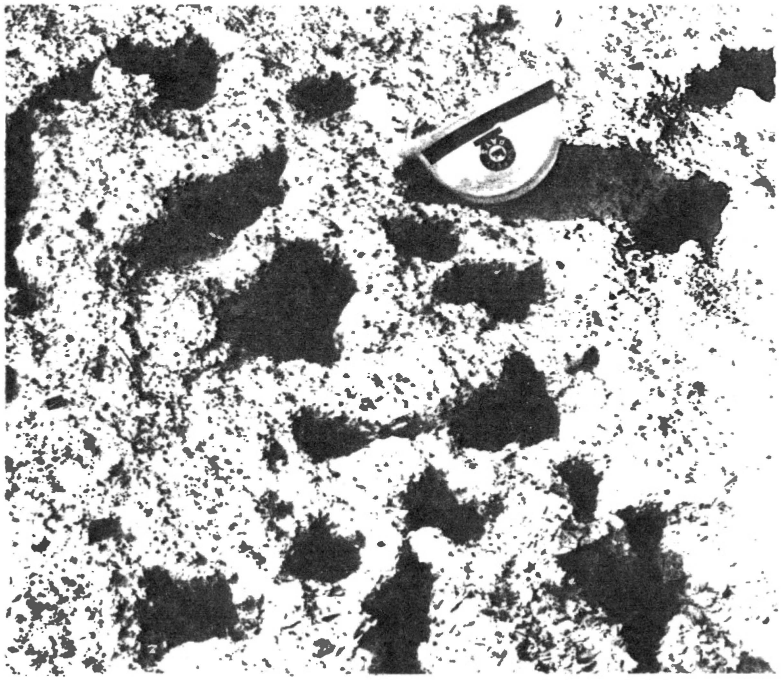
Source: (194)

Figure 129. Uncovered Upper End of Erosion Pipe in Dispersive Clay, Wister Dam.



Source: (184)

Figure 130. Vertical Erosion Tunnels in Dispersive Clay, Sardis Dam, Oklahoma.



Source: (184)

Figure 131. Pockmarking due to Vertical Erosion Tunnels, Sardis Dam, Oklahoma.

Identification and Treatment

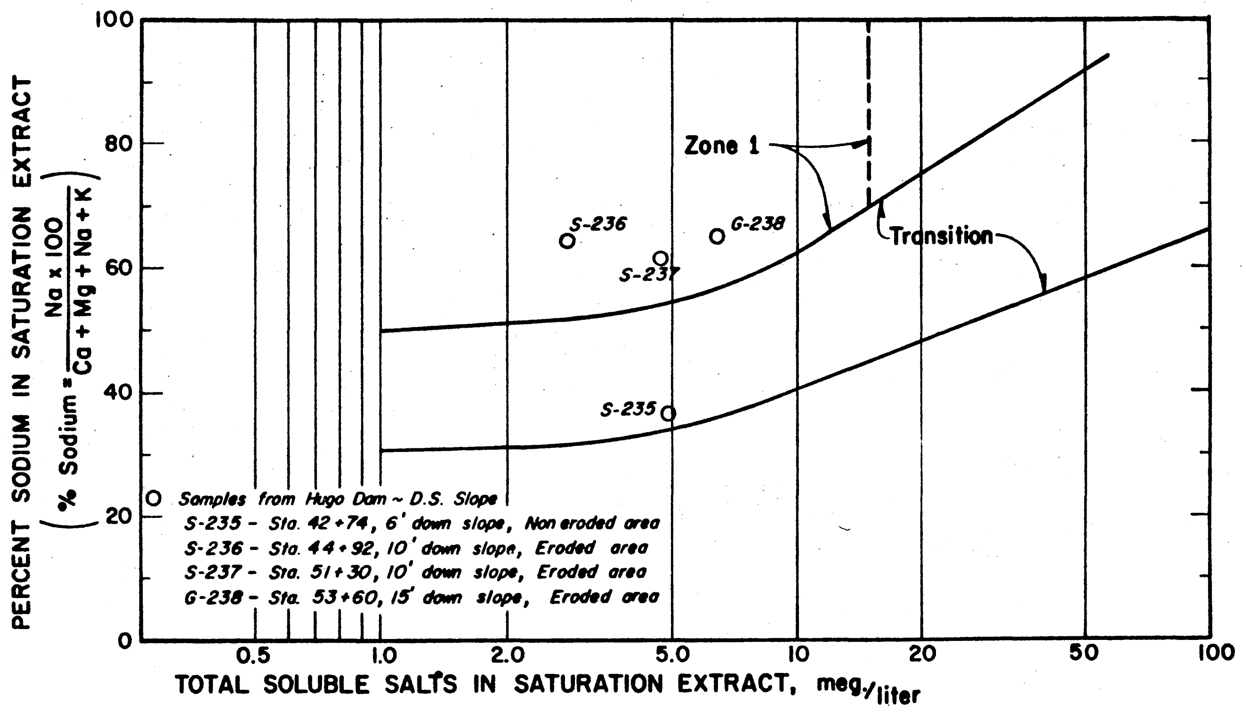
Three samples from an eroded area of the embankment slope at Hugo Dam in Oklahoma were subjected to chemical analysis along with a control sample taken from a noneroded area. The results are plotted in Figure 132. The three samples from the eroded area plotted within the zone of data from 16 dams previously damaged by dispersive clay erosion while the control sample plotted out of the dispersive zone (165).

At Hugo Dam, the eroded area was repaired using lime treatment of the surface soils and has provided satisfactory performance. At Wister Dam, treatment has consisted of excavating to the erosion tunnels and backfilling with granular materials and topsoil accompanied by seeding or sodding.

Hydraulic Fracturing

Mechanics

The mechanism by which hydraulic fracturing occurs requires the effective stress to become tensile and equal in magnitude to the tensile strength of the soil. Effective tensile stress may occur within a soil mass as a result of differential deformation of the mass with tensile stresses developing through bending, or as a result of a substantial increase in pore pressure. An increase in pore pressure results in three simultaneous effects: (1) the effective stress decreases; (2) a zone of wetting is created within which pore pressures may vary; and (3) the total stress may increase due to swelling or decrease due to wetting. The effective stress can only become tensile as a result of pore pressures acting as a wedge in the soil mass.



Zone I *includes all samples from 16 clay dams which were damaged by tunnel-erosion from rainfall in Venezuela, Oklahoma, Mississippi, Arkansas, Tennessee and Texas.*

Source: (165)

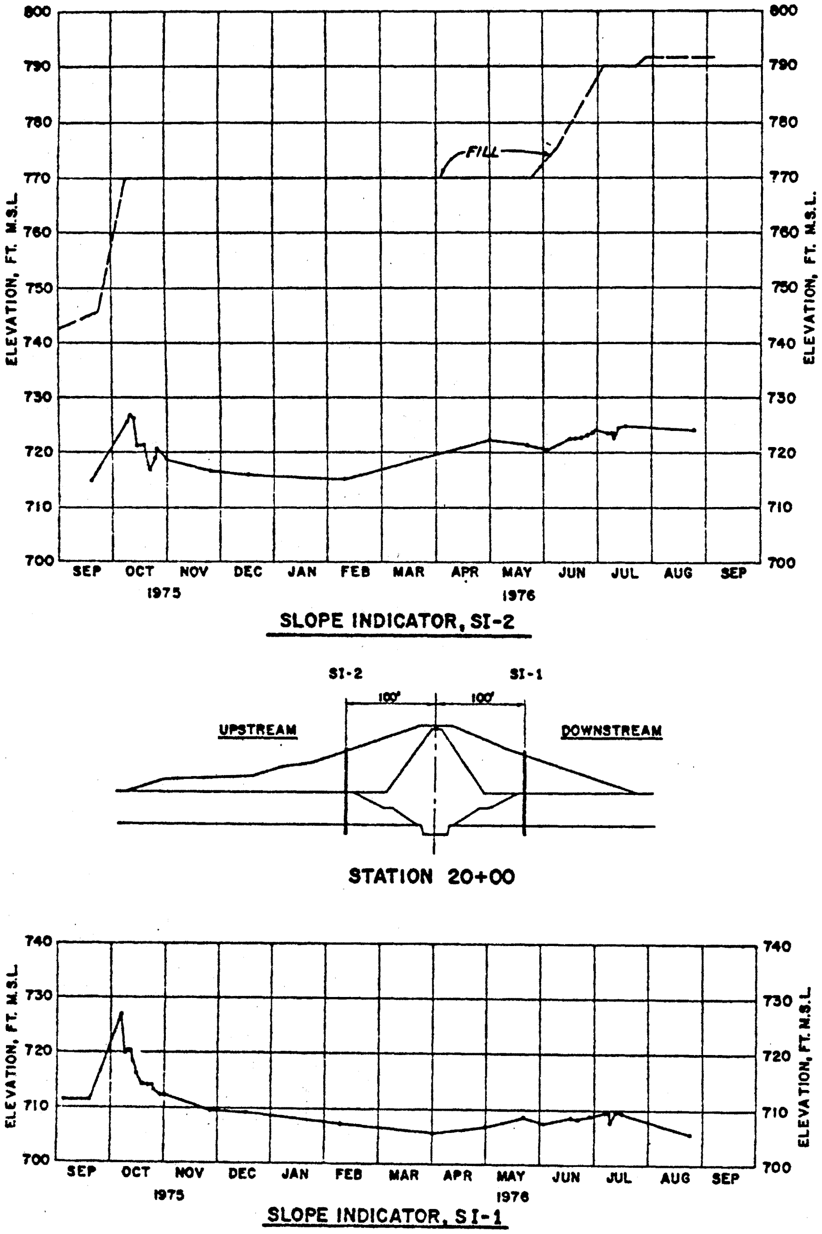
Figure 132. Results of Chemical Analysis of Soil Samples from Hugo Dam, Oklahoma.

Features which produce this type of wedging action include boreholes, pre-existing cracks, selective seepage paths, and zones of poor compaction within the earthfill structure. A uniform increase in pore pressure within a continuous and homogeneous soil mass will not result in tensile stresses and subsequent hydraulic fracturing (32).

Induced Fracturing

Hydraulic fracturing which accompanies design and construction discrepancies and results in embankment cracking has been presented in Chapter 3, Embankment Design, and is induced by reservoir filling. Construction loading during fill placement over soft clay foundations is sufficient to induce hydraulic fracturing as a result of massive construction pore pressures. The rapid construction of Birch Dam, upon 30 feet of soft clays, produced appreciable pore pressures which are believed to have contributed to the overthrust of the embankment, as shown in Figure 49 in Chapter 3. The rapid dissipation of observed pore pressures within the foundation at Birch Dam, shown in Figure 133, are believed to have been the result of loss of water within the inclinometer casing through cracks within the embankment (140).

A major concern during drilling operations through an earth embankment is the possibility of inducing hydraulic fracturing of the earthfill through the use of drilling fluids. Drilling through nearly saturated, plastic impervious material is particularly hazardous because of the possibility of squeezing of the hole over the drill bit thereby closing the discharge path of high pressure drilling fluids which carry cuttings to the surface. Drilling operations to install inclinometer casings in the embankment at Skiatook Dam in Oklahoma



SLOPE INDICATOR, SI-2

STATION 20+00

SLOPE INDICATOR, SI-1

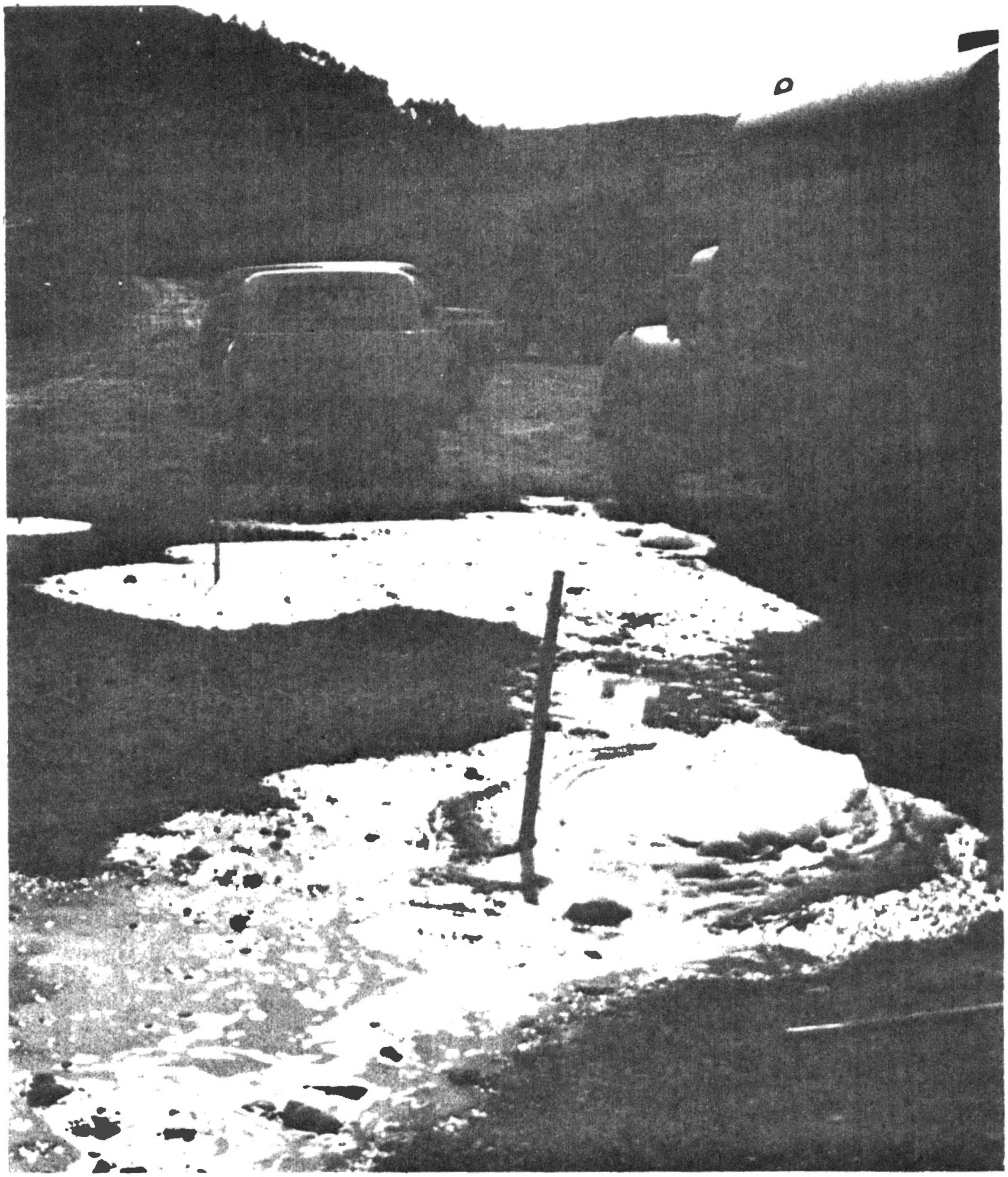
Source: (140)

Figure 133. Piezometric Response of Inclinometers Due to Construction Loading at Birch Dam.

resulted in considerable fracturing of the embankment as the drill holes squeezed closed. Drilling fluid was recovered from a previuosly installed inclinometer casing over 100 feet away (218). A similar situation occurred at Warm Springs Dam in California during drilling operations to install piezometers within the earthfill. The drilling fluid was under high pressures for effective removal of drill cuttings and, when the drill hole caved, substantial cracking of the embankment occurred as shown along the crest in Figure 134. After the piezometer installation was completed, a testpit was excavated adjacent to the piezometer riser to evaluate the extent of fracturing. Drilling fluid which had been retained in embankment fractures may be seen emitting into the testpit in Figure 135. Investigation confirmed that the hydraulic fracturing was limited only to the upper portion of the upstream randon shell and had not violated the integrity of the impervious core (123).

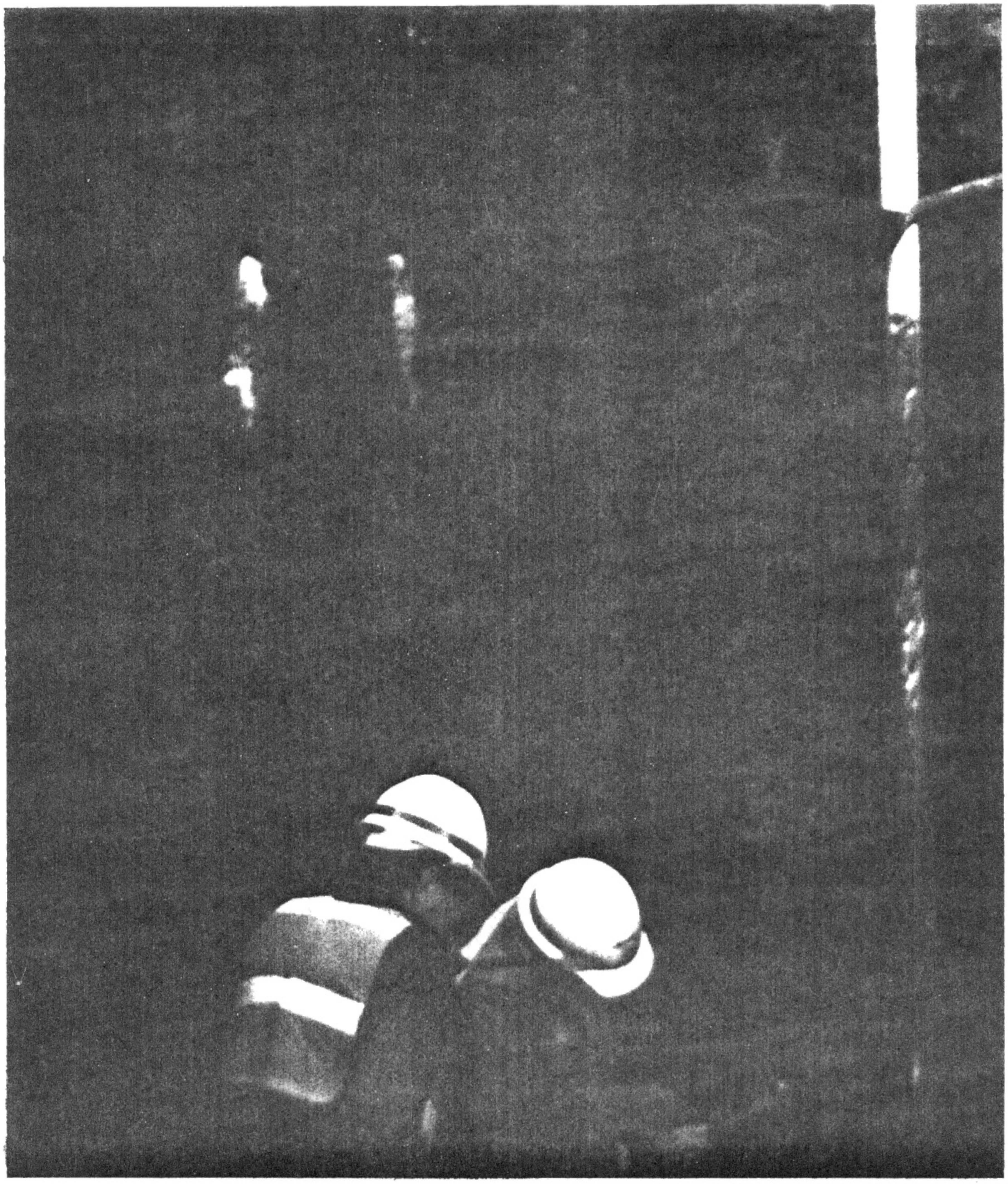
Preventive Measures

Hydraulic fracturing induced by an impounded reservoir pool can occur if discontinuities are present to allow reservoir water pressure to produce wedging action within the embankment. Therefore design and construction considerations must address the elimination of discontinuities which would allow wedging and subsequent hydraulic fracturing. Features which create discontinuities include loose rock fragments, open cracks in foundation rock, depressions in the foundation surface, and differential settlement cracks (33). Foundation preparation as outlined in Chapter 5 will alleviate the potential for creating discontinuities and hydraulic fracturing.



Source: (123)

Figure 134. Drilling Fluid Discharge at Crest due to Hydraulic Fracturing of Embankment during Piezometer Installation, Warm Springs Dam, California.



Source: (123)

Figure 135. Interior of Test Pit Excavated to Evaluate Extent of Hydraulic Fracturing, Warm Springs Dam.

Controlling construction rates of fill placement will control pore pressures. Piezometers would be required to monitor increases and dissipation of pore pressures. Compaction water contents near optimum to dry of optimum would reduce the potential for significant buildup of pore pressures.

Ideally, instrumentation should be installed in the foundation prior to fill placement and in the embankment during fill placement. However, supplemental instrumentation may be necessary to replace or verify existing instrumentation. Drilling operations which utilize drilling fluids or air should never be used within an earthfill embankment. Earth augers, although somewhat slower than rotary bits, remove cuttings through mechanical means, and the borehole can be maintained with temporary casing.

Collapsible Soils

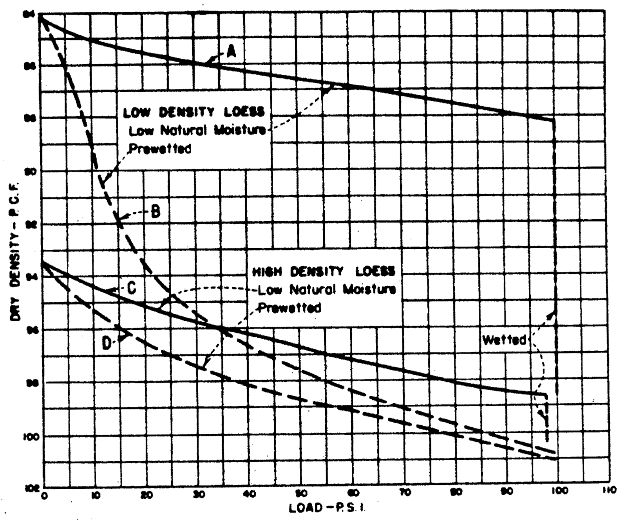
Type and Nature of Collapsible Soils

Collapsible soils are defined as any unsaturated soil which undergoes a radical rearrangement of particles and corresponding volume decrease upon wetting. The most extensive deposits of collapsible soils are aeolian sands and silts, or loess, and is found in large areas of the midwestern and western United States. Although less common but of a similar nature are alluvial fans, mud flows, flowslides, and residual soils from which leaching has removed soluble and fine-grained material (17).

The typical structure of collapsible soils consists of silt or sand particles in a honeycomb pattern with a cementing agent which is usually clay, at the points of contact of the individual particles. Upon wetting, the cementing agent becomes dissolved and the structure collapses. As long as the soil remains dry, this honeycomb structure can bear substantial loads. However, after completion of the embankment and impoundment of the reservoir, the honeycomb structure will soften and collapse when wetted for the first time by foundation underseepage. If proper measures are not undertaken to control excessive settlement of collapsible soils, failure of the dam may occur by one of three modes: (1) instantaneous, differential settlement causing rupture of the impervious core and breaching of the dam; (2) substantial settlement of the embankment and loss of freeboard causing overtopping of the dam; or (3) isolated settlement of the foundation accompanied by bridging of the embankment and piping of the embankment material.

Prewetting

The Bureau of Reclamation has constructed a number of earth dams on loess foundations, primarily within the Missouri River Basin. The procedure used by the Bureau involves prewetting of the foundation loess by ponding over flatter areas through the construction of dikes and by sprinkling steeper slopes. Typical compression curves for loess in the Missouri River Basin are presented in Figure 136 and clearly illustrate the advantages of prewetting to achieve densification of foundation loess. Hence, post-construction collapse of loess due to saturation by the reservoir can be avoided by prewetting the foundation in order to obtain compression during construction of the embankment (210).

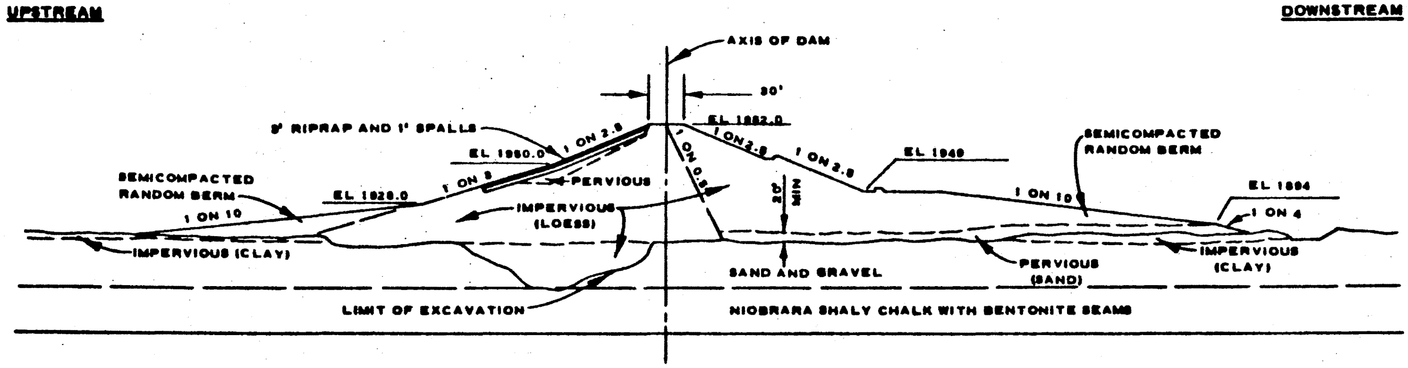


Source: (210)

Figure 136. Compression Curves of Loess in Missouri River Basin.

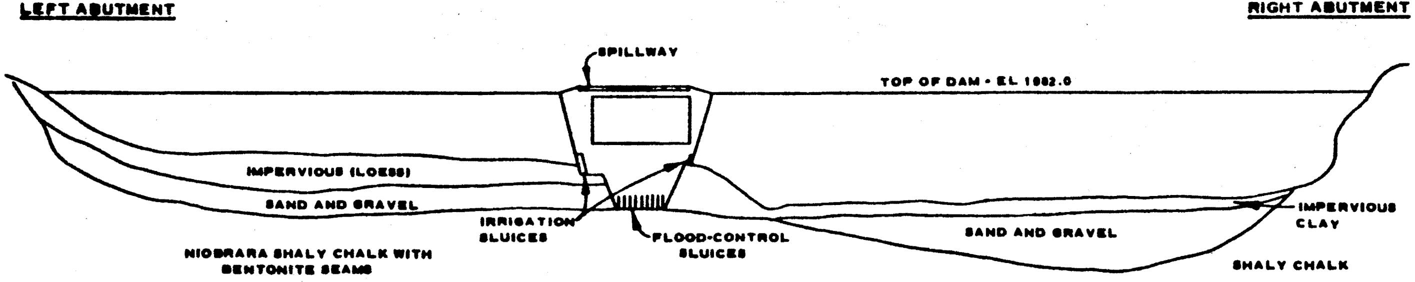
Excavation and Removal

If the prewetting fills the voids with water but does not dissolve the cementing agent and cause preconstruction collapse of the structure, liquefaction may occur when the shear strains from the embankment weight result in a sudden collapse of the structure (62). As a result of this potential for liquefaction, the Corps of Engineers excavated and recompacted up to 40 feet of loess from the foundation of Harlan County Dam in Nebraska (200). The typical embankment section and valley cross section at Harlan County Dam are shown in Figure 137. Because of the erodibility of the recompacted loess of the foundation, a 20-foot thick horizontal filter blanket was provided along the downstream portion of the embankment foundation.



NOTE: 1. BERMS USED RIGHT OF SPILLWAY ONLY.
2. FOUNDATION EXCAVATION EXTENDS BENEATH THE ENTIRE EMBANKMENT LEFT OF THE SPILLWAY.

TYPICAL EMBANKMENT SECTION



VALLEY CROSS SECTION

Source: (200)

Figure 137. Harlan County Dam, Republican River, Nebraska.

CHAPTER VIII

SUMMARY AND CONCLUSIONS

At the present time, the conceptual theory of soil mechanics and theoretical analyses have far exceeded the accuracy that can be justified based upon the knowledge of insitu and remolded soil properties and actual foundation conditions. Stability and seepage analyses place far too much emphasis on methodology at the expense of establishing suitable input parameters. Professional literature is saturated with methods of stability analysis which provide numerical solutions that vary less than 5 percent, while the accuracy of the individual strength parameters may vary more than 30 percent. However, in defense of new methodology, the phenomenon of cracking and hydraulic fracturing are immeasurably better understood due to pioneering efforts through finite element analyses.

Of the historical causes of dam failures (overtopping, internal erosion through piping, slope slides, and seepage along conduits), erosion through piping is presently the primary cause. Advanced techniques of hydrological analysis and hydraulic design has virtually eliminated overtopping as a possibility at major earth dams. Increased understanding of soil behavior and thorough compaction control have significantly reduced occurrences of slope instability and seepage along conduits. But erosion through piping, although reasonably understood in concept, continues to plague earth dams, due in part to the uncertainties associated with drawing inferences from foundation investigations. And, unfortunately, there is not enough distribution of knowledge gained through past experience.

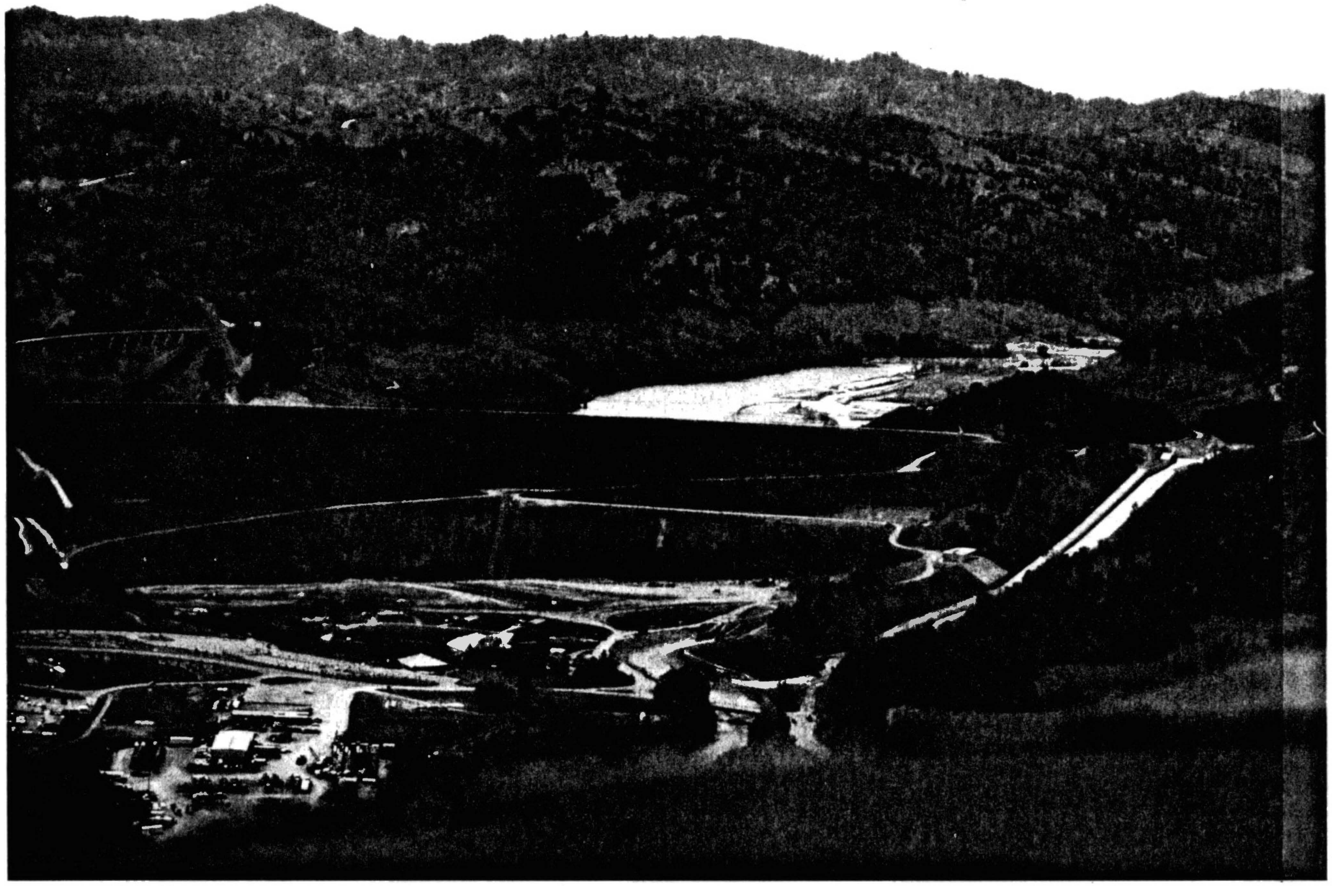
The treatment of foundation rock to reduce and control underseepage has made significant progress within the past decades primarily as a result of seepage problems at East Branch Dam and Wolf Creek Dam, and the failure of Teton Dam. The treatment of foundation rock does not involve providing increased stability of the rock, except through identification of problematic rock and possibly excavation and removal.

The concepts of construction on soft clay foundations are well understood but are rarely executed in practice until dramatic problems occur during construction. A good example of this was the construction of West Branch Dam which was nearly completed before overwhelming problems necessitated electroosmosis to dewater the foundation clays. This is also evidenced by the reluctance of engineers to install foundation instrumentation prior to construction; the attitude of many engineers is that there is no need for instrumentation unless problems occur.

Defensive design measures to accommodate foundation and embankment uncertainties should be incorporated, and include: (1) selection of design shear strengths which represent the lower range of test data to allow for foundation and embankment uncertainties; (2) selection of compressibility and seepage parameters within the upper range of data; (3) selection of permeabilities from the lower range for drains; and (4) safety factors for stability analyses that allow for margins of error in estimating as-built conditions.

To achieve design concepts, the design process for an earth dam should not be considered complete until the project is completed. Foundation investigations are greatly inhanced by inspection and evaluation during cutoff trench excavation or by grouting operations. The properties of compacted earthfill and rockfill are better understood through actual fill placement during construction. During the initial design phase and the construction phase, existing earth dams with similiar foundation and borrow properties should be studied to anticipate problems and postulate solutions.

Warm Springs Dam in California, Figure 138, is the most recently completed earth dam within the United States; the embankment was completed in 1983. The design and construction of Warm Springs Dam incorporated the very latest state-of-the-art design processing systems and construction monitoring system. Warm Springs Dam also incorporated the latest state-of-the-art empirical design features which were developed through experience which dates back to, and beyond, Fort Peck Dam.



Source: (123)

Figure 138. Warm Springs Dam - Lake Sonoma, California.

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