DESIGN OF HYDRAULIC STRUCTURES (4-0-0)

Module I

Concrete Dams: Investigation and Planning, Forces on Concrete dams, Types of loads, Stability analysis. Safety criteria, Gravity analysis, Internal stress calculation and Galleries. Joints and keys and cooling arrangement, Water stops at joint, closing gaps. Buttress and Arch Dam. Mass concrete for dams: Properties and quality control. Pressure grouting.

Selection of Type of Dam

A. CLASSIFICATION OF TYPES

4.1. Genera/.-Dams may be classified into a number of different categories, depending upon the purpose of the classification. For the purposes of this manual, it is convenient to consider three broad classifications: Dams are classified according to their use, their hydraulic design, or the materials of which they are constructed.

4.2. Classification According to Use.-Dams may be classified according to the broad function they serve, such as storage, diversion, or detention. Refinements of these classifications can also be made by considering the specific functions involved. Storage dams are constructed to impound water during periods of surplus supply for use during periods of deficient supply. These periods may be seasonal, annual, or longer. Many small dams impound the spring runoff for use in the dry summer season.

Storage dams may be further classified according to the purpose of the storage, such as water supply, recreation, fish and wildlife, hydroelectric power generation, irrigation, etc. The specific purpose or purposes to be served by a storage dam often influence the design of the structure and may establish criteria such as the amount of reservoir fluctuation expected or the amount of reservoir seepage permitted. Figure 4-l shows a small earth-fill storage dam, and figure 4-2 shows a concrete gravity structure serving both diversion and storage purposes.

Diversion dams are ordinarily constructed to provide head for carrying water into ditches, canals, or other conveyance systems. They are used for irrigation developments, for diversion from a live stream to an off-channel-location storage reservoir, for municipal and industrial uses, or for any combination of the above. Figure 4-3 shows a typical small diversion dam.

Detention dams are constructed to retard flood runoff and minimize the effect of sudden floods. Detention dams consist of two main types. In one type, the water is temporarily stored and released through an outlet structure at a rate that does not exceed the carrying capacity of the channel downstream.

In the other type, the water is held as long as possible and allowed to seep into pervious banks or into the foundation. The latter type is sometimes called a water-spreading dam or dike because its main purpose is to recharge the underground water supply. Some detention dams are constructed to trap sediments; these are often called debris dams.

Although it is less common on small projects than on large developments, dams are often constructed to serve more than one purpose. Where multiple purposes are involved, a reservoir allocation is usually made to each distinct use. A common multipurpose project combines storage, flood control, and recreational uses.

4.3. Classification by Hydraulic Design.-

Dams may also be classified as overflow or non-overflow dams.

Overflow dams are designed to carry discharge over their crests or through spillways along the crest. Concrete is the most common material used for this type of dam.

Non-overflow dams are those designed not to be overtopped. This type of design extends the choice of materials to include earth-fill and rock-fill dams.

Often the two types are combined to form a composite structure consisting of, for example, an overflow concrete gravity dam with earth-fill dikes.

Figure 4-4 shows such a composite structure built by the Bureau of Reclamation.

4.4 Classification by Materials.-The most common classification used for the discussion of design procedures is based upon the materials used to build the structure. This classification also usually recognizes the basic type of design, for example, the “concrete gravity” dam or the “concrete arch” dam.

This text is limited in scope to consideration of the more common types of dams constructed today; namely, earth-fill, rock-fill, and concrete gravity dams. Other types of dams, including concrete arch, concrete buttress, and timber dams, are discussed briefly with an explanation of why their designs are not covered in this text.

4.5. Earth-fill Dams.-Earth-fill dams are the most common type of dam, principally because their construction involves the use of materials from required excavations and the use of locally available natural materials requiring a minimum of processing. Using large quantities of required excavation and locally available borrow are positive economic factors related to an earth-fill dam. Moreover, the foundation and topographical requirements for earth-fill dams are less stringent than those for other types. It is likely that earth fill dams will continue to be more prevalent than other types for storage purposes, partly because the number of sites favorable for concrete structures is decreasing as a result of extensive water storage development. This is particularly true in arid and semiarid regions where the conservation of water for irrigation is a fundamental necessity.

Although the earth-fill classification includes several types, the development of modern excavating, hauling, and compacting equipment for earth materials has made the rolled-fill type so economical as to virtually replace the semi-hydraulic-fill and hydraulic-fill types of earth-fill dams. This is especially true for the construction of small structures, where the relatively small amount of material to be handled precludes the establishment of the large plant required for efficient hydraulic operations.

For these reasons, only the rolled-fill type of earth-fill dam is treated in this text. Rolled-fill Earth-fill dams are further classified as "homogeneous," "zoned," or "diaphragm," as described in chapter 6.

Earth-fill dams require appurtenant structures to serve as spillways and outlet works. The principal disadvantage of an earth fill dam is that it will be damaged or may even be destroyed under the erosive action of overflowing water if sufficient spillway capacity is not provided. Unless the site is off-stream, provision must be made for diverting the stream past the dam site through a conduit or around the dam site through a tunnel during construction. A diversion tunnel or conduit is usually provided for a concrete dam; however, additional provisions can be made for overtopping of concrete blocks during construction. A gap in an embankment dam is sometimes used for routing the river through the dam site during construction of portions of the dam on either or both sides of the gap. See chapter 11 for a more detailed description of diversion during construction.

4.6. Rock-fill Dams.-Rock-fill dams use rock of all sizes to provide stability and an impervious membrane to provide water tightness. The membrane may be an upstream facing of impervious soil, a concrete slab, asphaltic-concrete paving, steel plates, other impervious elements, or an interior thin core of impervious soil.

Like the earth embankments, rock-fill dams are subject to damage or destruction by the overflow of water and so must have a spillway of adequate capacity to prevent overtopping. An exception is the extremely low diversion dam where the rock-fill facing is designed specifically to withstand overflows.

Rock-fill dams require foundations that will not be subject to settlements large enough to rupture the watertight membrane. The only suitable foundations, therefore, are rock or compact sand and gravel.

The rock-fill type dam is suitable for remote locations where the supply of good rock is ample, where the scarcity of suitable soil or long periods of high rainfall make construction of an earth-fill dam impractical, or where the construction of a concrete dam would be too costly. Rock-fill dams are popular in tropical climates because their construction is suitable for long periods of high rainfall.

4.7. Concrete Gravity Dams.-Concrete gravity dams are suitable for sites where there is a reasonably sound rock foundation, although low structures may be founded on alluvial foundations if adequate cutoffs are provided. They are well suited for use as overflow spillway crests and, because of this advantage, are often used as spillways for earth-fill or rock-fill dams or as overflow sections of diversion dams.

Gravity dams may be either straight or curved in plan. The curved dam may offer some advantage in both cost and safety. Occasionally the dam curvature allows part of the dam to be located on a stronger foundation, which requires less excavation. The concept of constructing concrete dams using RCC (roller-compacted concrete) has been developed and implemented. Several RCC dams have been constructed in the United States and in other countries. The technology and design procedures, however, are not presented in this manual because procedures and approaches are relatively new and are still being developed.

4.8. Concrete Arch Dams.-Concrete arch dams are suitable for sites where the ratio of the width between abutments to the height is not great and where the foundation at the abutments is solid rock capable of resisting arch thrust. Two types of arch dams are defined here: the single and the multiple arch dam. A single arch dam spans a canyon as one structure and is usually limited to a maximum crest length to height ratio of 10:1. Its design may include small thrust blocks on either abutment, as necessary, or a spillway somewhere along the crest. A multiple arch dam may be one of two distinct designs. It may have either a uniformly thick cylindrical barrel shape spanning 50 feet or less between buttresses, such as Bartlett Dam in Arizona, or it may consist of several single arch dams supported on massive buttresses spaced several hundred feet on centers. The dam's purpose, whether it be a permanent major structure with a life expectancy of 50 years or a temporary cofferdam with a useful life of 5 years, will directly influence the time for design and construction, the quality of materials in the dam and foundation, the foundation treatment, and the hydraulic considerations. Structural and economic aspects prohibit the design of an arch dam founded on stiff soil, gravel, or cobblestones.

Uplift usually does not affect arch dam stability because of the relative thinness through the section, both in the dam and at the concrete rock contact.

Historically, both permanent and temporary concrete dams have survived partial and complete inundation, both during and after construction.

Because the design of an arch dam is specialized, a detailed discussion is not included in this book.

Refer to Design of Arch Dams, a Bureau of Reclamation publication, for discussions on design, loads, methods of analysis, safety factors, etc.

4.9. Concrete Buttress Dams.-Buttress dams are comprised of flat deck and multiple arch structures.

They require about 60 percent less concrete than solid gravity dams, but the increased formwork and reinforcement steel required usually offset the savings in concrete. A number of buttress dams were built in the 1930’s, when the ratio of labor costs to material costs was comparatively low. The cost of this type of construction is usually not competitive with that of other types of dams when labor costs are high.

The design of buttress dams is based on the knowledge and judgment that comes only from specialized experience in that field. Because of this fact and because of the limited application for buttress dams under present-day conditions, their design is not covered in this text.

4.10. Other Types. -Dams of types other than those mentioned above have been built, but in most cases they meet some unusual local requirement or are of an experimental nature. In a few instances, structural steel has been used both for the deck and for the supporting framework of a dam. And before

1920, a number of timber dams were constructed, particularly in the Northwest. The amount of labor involved in the timer dam, coupled with the short life of the structure, makes this type of structure uneconomical for modern construction. Timber and other uncommon types of dams are not treated in this text.

B. PHYSICAL FACTORS GOVERNING SELECTION OF TYPE

4.11. Genera/.-During the early stages of planning and design, selection of the site and the type of dam should be carefully considered, It is only in exceptional circumstances that only one type of dam or appurtenant structure is suitable for a given dam site. Generally, preliminary designs and estimates for several types of dams and appurtenant structures are required before one can be proved the most suitable and economical. It is, therefore, important to understand that the project is likely to be unduly expensive unless decisions regarding the site selection and the type of dam are based upon adequate study.

The selection of the type of dam requires cooperation among experts representing several disciplines-including planners; hydrologists; geotechnical, hydraulic, and structural engineers; and engineering geologists-to ensure economical and appropriate designs for the physical factors, such as topography, geology and foundation conditions, available materials, hydrology, and seismicity.

Protection from spillway discharges, limitations of outlet works, the problem of diverting the stream during construction, availability of labor and equipment, accessibility of the site, physical features of the site, the purpose of the dam, and dam safety all affect the final choice of the type of dam. Usually, the final choice of the type of dam is based on a comparison of the costs to construct the various dam types studied. The following paragraphs discuss important physical factors in the choice of the type of dam.

4.12. Topography.-Topographic considerations include the surface configuration of the dam site and of the reservoir area and accessibility to the site and to construction materials. Topography, in large measure, dictates the first choice of the type of dam. A narrow stream flowing between high, rocky walls would naturally suggest a rock-fill or concrete overflow dam. On the other hand, low, rolling plains would suggest an earth-fill dam. Intermediate conditions might suggest other choices, such as a composite structure. The point is that topography is of major significance in choosing the dam type.

Topography may also have an important influence on the selection of appurtenant structures. For example, if there are natural saddles, it may be possible to locate a spillway through a saddle. If the reservoir rim is high compared with the dam height, and it is unbroken, a chute or tunnel spillway might be necessary. The spillway considerations can influence the type of dam. In a deep, steep-walled canyon, it might be more economical to construct a concrete dam with an overflow spillway than to provide a spillway for a rock-fill dam.

4.13. Geology and Foundation Conditions. -

The suitability of the various types of rock and soil as foundation and construction materials are geologic questions that must be considered. The foundation geology at a dam site often dictates the type of dam suitable for that site. The strength, thickness, and inclination of strata; permeability; fracturing; and faulting are all important considerations in selecting the dam type. Some of the different foundations commonly encountered are discussed below.

(a) Rock Foundations,-Competent rock foundations, which are free of significant geologic defects, have relatively high shear strengths, and are resistant to erosion and percolation, offer few restrictions as to the type of dam that can be built upon them. The economy of materials or the overall cost should be the ruling factor. The removal of disintegrated rock together with the sealing of seams and fractures by grouting is frequently necessary.

Weaker rocks such as clay shales, some sandstones, weathered basalt, etc., may present significant problems to the design and construction of a dam and may heavily influence the type of dam selected.

(b) Gravel Foundations.-Gravel foundations, if well compacted, are suitable for earth-fill or rock-fill dams. Because gravel foundations are frequently subjected to water percolation at high rates, special precautions must be taken to provide adequate seepage control or effective water cutoffs or seals.

(c) Silt or Fine Sand Foundations.-Silt or fine sand foundations can be used for low concrete gravity dams and earth-fill dams if properly designed, but they are generally not suitable for rock-fill dams.

Design concerns include non-uniform settlement, potential soil collapse upon saturation, uplift forces, the prevention of piping, excessive percolation losses, and protection of the foundation at the downstream embankment toe from erosion.

(d) Clay Foundations.-Clay foundations can be used for the support of earth-fill dams, but require relatively flat embankment slopes because of relatively lower foundation shear strengths. Clay foundations under dams can also consolidate significantly.

Because of the requirement for flatter slopes and the tendency for clay foundations to settle a lot, it is usually not economical to construct a rock-fill dam on a clay foundation. Clay foundations are also ordinarily not suitable for concrete gravity dams. Tests of the foundation material in its natural state are usually required to determine the consolidation characteristics of the foundation strata and their ability to support the superimposed load.

(e) Non-uniform Foundations.-Occasionally, situations occur where reasonably uniform foundations of any of the types described above cannot be found and where a non-uniform foundation of rock and soft material must be used if the dam is to be built. Nevertheless, such conditions can often be counterbalanced by special design features. Even dam sites those are not highly unusual present special problems requiring the selection of appropriate treatment by experienced engineers.

The details of the foundation treatments mentioned above are given in the appropriate chapters on the design of earth-fill, rock-fill, and concrete gravity dams .

4.14. Materials Available. -Materials for dams of various types that may sometimes be available at or near the site are:

- Soils for embankments
- Bock for embankments and riprap
- Concrete aggregate (sand, gravel, crushed stone)

Elimination, or reduction of transportation expenses for construction materials, particularly those used in great quantities, reduces the total cost of the project considerably. The most economical type of dam is often the one for which a large quantity of materials can be found within a reasonable distance from the site.

The availability of suitable sand and gravel for concrete at a reasonable cost locally and, perhaps, even on property to be acquired for the project is a factor favorable to the selection of a concrete structure.

The availability of suitable rock for rock-fill is a factor favorable to the selection of a rock-fill dam.

Every local resource that reduces the cost of the project without sacrificing the efficiency and quality of the final structure should be used.

4.15. Hydrology,-Hydrologic studies examine the project purposes stated in section 4.2 in the paragraph on storage dams. There is a close relationship between the hydrologic and economic factors governing the choice of the type of dam and appurtenant structures. Stream flow characteristics and precipitation may appreciably affect the cost of construction by influencing the treatment and diversion of water and extending the construction time. Where large tunnels are required for diversion, conversion of the tunnels to tunnel spillways may provide the most economical spillway alternative.

4.16. Spillway. -A spillway is a vital appurtenance of a dam. Frequently, its size and type and the natural restrictions in its location are the controlling factors in the choice of the type of dam.

Spillway requirements are dictated primarily by the runoff and stream flow characteristics, independent of site conditions or type or size of the dam. The selection of specific spillway types should be influenced by the magnitudes of the floods to be passed.

Thus, it can be seen that on streams with large flood potential, the spillway is the dominant structure, and the selection of the type of dam could become a secondary consideration.

The cost of constructing a large spillway is frequently a considerable portion of the total cost of the project. In such cases, combining the spillway and dam into one structure may be desirable, indicating the selection of a concrete overflow dam.

In certain instances, where excavated material from separate spillway channels can be used in the dam embankment, an earth-fill dam may prove to be advantageous.

Small spillway requirements often favor the selection of earth-fill or rock-fill dams, even in narrow dam sites.

The practice of building overflow concrete spillways on earth or rock embankments has generally been discouraged because of the more conservative design assumptions and added care needed to forestall failures. Inherent problems associated with such designs are unequal settlements of the structure caused by differential consolidations of the embankment and foundation after the reservoir loads are applied; the need for special provisions to prevent the cracking of the concrete or opening of joints that could permit leakage from the channel into the fill, with consequent piping or washing away of the surrounding material; and the requirement for having a fully completed embankment before spillway construction can be started.

Consideration of the above factors coupled with increased costs brought about by more conservative construction details, such as arbitrarily increased lining thickness, increased reinforcement steel, cutoffs, joint treatment, drainage, and preloading, have generally led to selection of alternative solutions for the spillway design. Such solutions include placing the structure over or through the natural material of the abutment or under the dam as a conduit.

One of the most common and desirable spillway arrangements is the use of a channel excavated through one or both of the abutments outside the limits of the dam or at some point removed from the dam. Where such a location is adopted, the dam can be of the non-overflow type, which extends the choice to include earth-fill and rock-fill structures.

Conversely, failure to locate a spillway site away from the dam requires the selection of a type of dam that can include an overflow spillway. The overflow spillway can then be placed so as to occupy only a portion of the main river channel, in which case the remainder of the dam could be either of earth, rock, or concrete. Olympus Dam (fig. 4-4) is an example of this type of dam.

4.17. Earthquake.-If the dam lies in an area that is subject to earthquake shocks, the design must provide for the added loading and increased stresses. Earthquake design considerations for earth-fill, rock-fill, and concrete gravity dams are discussed in chapters 6, 7, and 8, respectively. For earthquake areas, neither the selection of type nor the design of the dam should be undertaken by anyone not experienced in this type of work.

Concrete Gravity Dams

A. INTRODUCTION

8.1. Origin and Development.--A concrete gravity dam is proportioned so that its own weight provides the major resistance to the forces exerted upon it. If the foundation is adequate and the dam is properly designed and constructed, the concrete dam will be a permanent structure that requires little maintenance.

Gravity dams of un-cemented masonry were built several thousand years B.C. Evidence found in archeological sites indicate dam base widths as much as four times the height. With the passing of centuries, various types of mortar have been used to bind the masonry, thereby increasing stability and water tightness and permitting steeper slopes to be used. Concrete and cement mortar were used in the construction of cyclopean masonry dams, the forerunners of the modern mass concrete gravity dams.

As an alternative to the conventional method of placing block upon block of mass concrete, RCC (roller-compacted concrete) is fast becoming an accepted method of constructing concrete gravity dams. An RCC dam is constructed in much the same way as an embankment dam. Zero slump concrete is placed, spread, and compacted with vibratory rollers in l- to 2-foot-thick lifts that are continuous between abutments. Because the RCC construction method is quicker and requires less labor, it is more cost efficient than conventionally placed mass concrete. Some of the concerns associated with the RCC construction method are bond strength and permeability along lift surfaces, cooling requirements, and incorporating transverse contraction joints. Because experience with RCC is still limited, improvements and changes are anticipated.

Upper Stillwater Dam, currently (1986) under construction, is the Bureau of Reclamation's first RCC dam.

8.2. Scope of Discussion.--In general, the discussion in this chapter applies to concrete gravity dams of any height. However, for dams much higher than 50 feet, the reader is referred to another Bureau of Reclamation publication [11] for additional details and considerations. This publication [l] is also beneficial to the designer of smaller dams and should be referenced in conjunction with the discussion contained herein.

This chapter discusses concrete properties, the forces that act on concrete gravity dams, foundation considerations, requirements for stability, and stress and stability analyses. Additional considerations for concrete structures on pervious (soil-like) foundations are presented, and current practices regarding miscellaneous details of design are briefly described. A brief discussion of current Bureau of Reclamation computer -methods is also included.

B. CONCRETE PROPERTIES

8.3. Strength.-A gravity dam should be constructed of concrete that will meet the design criteria for strength, durability, permeability, and other required properties. Properties of concrete vary with age, the type of cement, aggregates, and other ingredients, and their proportions in the mix.

Because different concretes gain strength at different rates, laboratory tests must be made on specimens of sufficient age to permit evaluation of ultimate strengths.

Normally, the concrete mix for gravity dams is designed for only compressive strength. However, compression is not the critical stress. Generally, a 10:l compressive strength to stress ratio results when designing the dam to meet the concrete shear and tensile strength limits. Therefore, tensile and shear strength are the most important concrete strength design parameters, and laboratory tests should be made to determine these values, especially across lift surfaces.

8.4. Elastic Properties.-Elastic properties are useful for analyzing deformations related to differential block movement, three-dimensional analyses, and other aspects concerned with deformations.

The modulus of elasticity, although not directly proportional to concrete strength, does increase with increasing concrete strength. As with the strength properties, the modulus of elasticity is influenced by mix proportions, cement, aggregate, admixtures, and age. The deformation that occurs immediately with the application of a load, such as during an earthquake, depends on the dynamic modulus of elasticity. The increase in deformation caused by a constant load over a period of time is the result of creep or plastic flow in the concrete.

The effects of creep are generally accounted for by determining a sustained modulus of elasticity of the concrete for use in the analyses of static loadings.

The static modulus of elasticity and Poisson's ratio should be determined for the different ages of concrete when test cylinders, made before or during construction, are loaded to failure within a few minutes according to standard ASTM loading rates.

The sustained modulus of elasticity under constant load should be determined from these cylinders after specific incremental loading periods for up to 1 and 2 years. The cylinders tested should be the same size and cured in the same manner as those used for compressive strength tests. The values of static modulus of elasticity, Poisson's ratio, and sustained modulus of elasticity used in the analyses should be the average of all test cylinder values.

8.5 Thermal Properties.-During construction, heat from cement hydration should be uniformly dissipated or controlled to avoid undesirable cracking. Uniform dissipation is accomplished by circulating cool water through tubing optimally spread atop each lift during conventional construction of vertical blocks. In addition, the heat generated can be reduced by replacing a portion of the cement with pozzolan, which generates only about 50 percent of the heat generated by the same quantity of cement.

Operational temperature changes from ambient air and the reservoir may produce steep nonlinear thermal gradients and associated stresses because of the slower response in the interior of the dam.

The thermal properties necessary for the evaluation of temperature changes are the coefficient of thermal expansion, thermal conductivity, specific heat, and diffusivity. The coefficient of thermal expansion is the length change per unit length for a 1°F temperature change. Thermal conductivity is the rate of heat conduction through a unit thickness over a unit area of the material subjected to a unit temperature difference between faces. The specific heat is defined as the amount of heat required to raise the temperature of a unit mass of the material 1°F. Diffusivity of concrete is an index of the ease with which concrete undergoes temperature change.

The diffusivity is calculated from the values of specific heat, thermal conductivity, and density.

8.4. Average Properties.-(a) Basic Considerations.- Concrete properties may be estimated from published data for preliminary studies until laboratory test data are available.

(b) Criteria.-The following average values may be used for preliminary designs until site-specific test data are available. Static values represent estimated values from laboratory tests for specimens loaded to failure within a few minutes according to standard ASTM loading rates.

- Compressive strength (static): 3,000 to 5,000 lb/in2
- Tensile strength (static): 5 to 6 percent of the compressive strength
- Tensile strength (dynamic): 10 percent of the static compressive strength
- Shear strength (static):
- Cohesion: 10 percent of the static compressive strength
- Coefficient of internal friction: 1.0
- Poisson's ratio: 0.2
- Static modulus of elasticity: 5.0 X 10fi lb/in2
- Dynamic modulus of elasticity: 6.0 X 10' lb/in2
- Sustained modulus of elasticity: 3.0 X 10fi lb/in2
- Coefficient of thermal expansion: 5.0 X 10mG ft/ft/"F
- Unit weight: 150 lb/ft"
- Diffusivity: 0.05 ft2/hr

C. FORCES ACTING ON THE DAM

8.7. Genera/.-Essential to the design of gravity dams is knowledge of the forces expected to affect the stresses and stability of the structure. The forces that must be considered are those due to (1) external water pressure, (2) temperature, (3) internal water pressure; i.e., pore pressure or uplift in the dam and foundation, (4) weight of the structure, (5) ice pressure, (6) silt pressure, (7) earthquake, and (8) forces from gates or other appurtenant structures.

Figure 8-l (A) shows the reservoir and tail water reactions on a non-overflow section. Symbols and definitions for this loading are:

Ψ = angle between face of the dam and the vertical, T = horizontal distance between the upstream and downstream faces of a section, I = moment of inertia of the base of a horizontal section 1 foot wide about its center of gravity, equal to T"/12, WC = weight of concrete, w = unit weight of water, 62.4 lb/ft3 h or h' = vertical distance from reservoir water or tail water, respectively, to base of section, P or P' = reservoir water or tail water pressure, respectively, at base of section, equal to wh or wh', W, = dead load weight above base of section under consideration including the weight of the concrete, WC, plus such appurtenances as gates and bridges, W, or W,l = vertical component of reservoir water or tail water load, respectively, on face above base of section, MO = moment of W, about center of gravity of base of section, Mw or M,' = moment of Ww or W,' about center of gravity of base of section, V or V' = horizontal component of reservoir water or tail water load, respectively, on face above base section, equal to wh"/2 for V and ~(h')~/2 for V' for normal conditions, MI, or Mp' = moment of V or V' about center of gravity of base of section, equal to wh3/6 for M, and w(tij3/6 for M,‘,

∑W = resultant vertical force above base of section, I3 = resultant horizontal force above base of section,

∑M = resultant moment from forces above base of section about center of gravity of base of section, e = distance from center of gravity of base of section to point where resultant of ∑W and ∑V intersects base of section, equal to .∑M/∑W, and U = total uplift force on horizontal section without drains or drains inoperable, equal to T(P+P')/2.

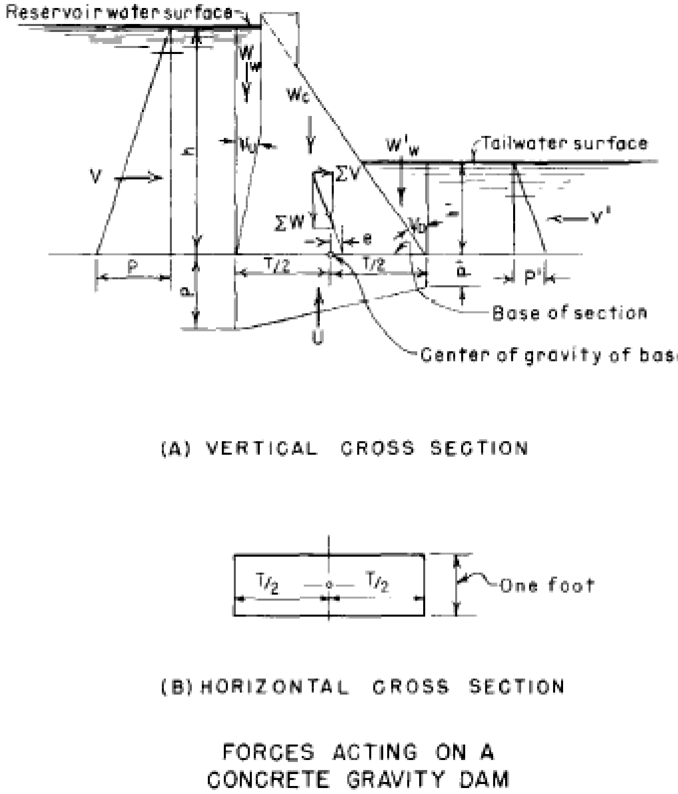


Figure 8-1.—Forces acting on a concrete gravity dam. 288-D-2505.

The summations of horizontal and vertical components imply a horizontal foundation or potential failure plane. Loads should be resolved into components normal and parallel to the foundation or to potential failure planes having significant slope in computing sliding stability.

8.8. External Water Pressure.-(a) Basic Considerations. - Reservoir and tail water loads are obtained from reservoir operation studies and tail water curves. These studies are based on operating and hydrologic data such as reservoir capacity, storage allocation, stream flow records, flood hydrographs, and reservoir releases. Reservoir operation curves derived from these studies reflect a normal high water surface, seasonal drawdowns, and the usual low water surface. See figure 8-2 for the following water surface designations.

(1) Maximum water surface.-The highest acceptable water surface elevation considering all factors affecting the safety of the structure. Normally, it is the maximum design reservoir elevation anticipated and usually occurs in conjunction with routing of the IDF (inflow design flood) through the reservoir. Maximum water surface usually corresponds to the dam crest elevation without including the parapet.

(2) Top of exclusive flood control capacity. - The reservoir water surface elevation at the top of the reservoir capacity allocated to exclusive use of regulating flood inflows to reduce damage downstream.

(3) Maximum controllable water surface elevation. - The highest reservoir water surface elevation at which gravity flows from the reservoir can be completely shut off. Generally, this is the top of the spillway gates or the crest of an ungated spillway.

(4) Top of joint-use capacity.-The reservoir water surface elevation at the top of the reservoir capacity allocated to joint uses of flood control and conservation purposes.

(5) Top of active conservation capacity.-The reservoir water surface elevation at the top of the capacity allocated to storage of water for conservation purposes only.

(6) Top of inactive capacity.-The reservoir water surface elevation below which the reservoir will not be evacuated under normal conditions.

(7) Top of dead capacity.-The lowest elevation in the reservoir from which water can be drawn by gravity.

(8) Streambed at the dam axis.-The elevation of the lowest point in the stream bed at the axis of the dam before construction.

This elevation normally defines the zero for area-capacity tables.

The normal design reservoir elevation is the top of joint-use capacity, if joint-use capacity is included. If not, it is the top of active conservation capacity.

On overflow dams without control features, the total horizontal water pressure on the upstream face is closely represented by the trapezoid (abed) on figure 8-3, in which the unit pressures at the top and bottom are wh, and wh, respectively, with w being the unit weight of water. The total horizontal force, P, passes through the center of gravity of the trapezoid. The vertical pressure component of water flowing over the top of the spillway is not used in the analysis because most of the total head has changed to velocity head. The sheet of water flowing down the downstream face generally does not exert enough pressure on the dam to warrant consideration. Where tail water or backwater stands against the downstream face, it should be treated in the same manner as the tail water on figure 8-1(A). However, during major overflows of water, the tail water pressures are involved in the energy dissipating process and may contribute only minor stabilizing forces on the dam.

(b) Criteria.-Reservoir elevations for the loading combinations analyzed should be selected from reservoir operation studies. The minimum tail water level associated with each reservoir level should be used. Tail water surface elevations should be obtained from tail water curves associated with operating studies. For computation of the reservoir and tail water loads, water pressure is considered to vary directly with depth and to act equally in all directions.

8.9. Temperature.-(a) Basic Considerations.- Volumetric increases caused by temperature rise transfer load across transverse contraction joints if the joints are grouted. The horizontal thrusts produced by volumetric changes associated with temperature increases result in a transfer of load across grouted contraction joints that increase the twisting effects and the loading of the abutments as discussed in [11]. Similarly, ungrouted contraction joints transfer horizontal thrusts at areas that come into contact when the concrete temperature exceeds the temperature necessary to close the contraction joint.

Temperature effects can also induce cracking in mass concrete structures. Tensile stresses that exceed the concrete tensile strength may be generated because of a restraint against temperature-induced volumetric changes. Temperature cracking can be prevented or greatly reduced by controlling the placement temperatures, the placement schedule, and the cooling of the mass concrete placed. The first two measures are usually sufficient to control cracking in small dams because the concrete dimensions are often thin enough to allow rapid dissipation of heat. For more details, see [a]. In addition, "Concrete Dam Design Standards," currently (1986) being prepared, will address the temperature considerations associated with mass concrete structures.

When the designer is making studies to determine concrete temperature loads, varying weather conditions should be considered. Similarly, a widely fluctuating reservoir water surface will affect the concrete temperatures. In determining temperature loads, the following conditions and temperatures should be used:

- Usual weather conditions.-The combination of three items that accounts for temperatures that are halfway between the mean monthly air temperatures and the minimum/maximum recorded air temperatures at the site [2]. The three items are (1) the daily air temperatures, (2) a l-week cycle representative of the cold/hot periods associated with barometric pressure changes, and (3) the mean monthly air temperatures.
- Usual concrete temperatures.-The usual concrete temperatures between the upstream and downstream faces are the average of the usual air temperatures and reservoir water

temperatures associated with the design reservoir operation. Additional refinement is obtained by considering the effects of solar radiation [a].

(b) Criteria-The effects of temperature change should always be investigated when joints are to be grouted and if the operating temperatures are above the closure temperature when joints are not to be grouted. The possibility of temperature-induced cracking should also be investigated.

8.10. Internal Water Pressures.-(a) Basic Considerations.-Water pressures caused by reservoir water and tail water occur within the dam and foundation in pores, cracks, joints, and seams. The distribution of internal water pressures along a horizontal section through the dam or its foundation is assumed to vary linearly from full reservoir pressure at the upstream face to zero or tail water pressure at the downstream face in the absence of drains or a more detailed analysis.

The internal water pressure, also called uplift, acts to reduce the compressive stresses normal to a horizontal section through the dam. Including a line of vertical formed drains within the dam and parallel to the upstream face serves to reduce the uplift force. The uplift reduction is dependent on the size, location, and spacing of the drains.

The generally accepted current practice assumes pore pressures act over 100 percent of the area of any section through the concrete. Current Bureau of Reclamation practice locates the line of drains at a distance from the upstream face equal to 5 percent of the maximum reservoir depth at the dam or at the same distance from the upstream face as the drains formed within contraction joints. Current Bureau practice assumes that a line of 5inchdiameter formed drains spaced 10 feet apart reduces the average pore pressure at the line of drains to tail water pressure plus one-third the differential between tail water and headwater pressures. These values are based on the assumption that the lowest elevation in the drainage gallery is at or below tail water level or that pumping of the drains will be a part of the operating criteria. If the gallery is above tail water elevation, the pressure at the line of drains should be determined as though the tail water level is equal to the gallery elevation. In no case should these pressures exceed those computed for the dam without drains. Internal pressures are assumed to be unaffected by earthquake accelerations because of the transitory nature of such accelerations. Forces from water pressures also occur within the foundation. Uplift forces in the foundation decrease the normal forces occurring on potential sliding planes. Water forces occurring on high angle joints increase driving forces on foundation blocks. Both of these reduce foundation sliding stability.

The uplift forces within the foundation and along the foundation-dam contact can be reduced by a line of drain holes drilled into the foundation from the floor of the foundation gallery. The internal pressure distribution through the foundation depends on depth, location, and orientation of the drains, rock permeability characteristics, jointing, faulting, and any other geologic features that may modify the flow. The line of drains should be located a distance downstream from the upstream dam face that will ensure that direct connection from the reservoir will not occur.

Determination of such pressure distributions can be made from flow nets computed by several methods, including two- and three-dimensional physical models, two- and three-dimensional finite element models, electric analogs, and graphical techniques.

For preliminary designs, the pressure at the line of drains can be estimated using the same approximation mentioned for formed drains within the dam. Basically, the pressure at the line of drains equals tail water pressure plus one-third the differential between headwater and tail water pressures.

This uplift assumption is generally conservative when the drain holes are drilled to a depth equal to 40 to 50 percent of the dam height and when the geologic conditions are uniform. Foundation drainage curtains generally consist of 3-inch-diameter holes drilled on lo-foot centers.

Uplift pressures under a concrete dam on a pervious (soil-like) foundation are related to seepage through permeable materials. Water percolating through pore spaces in these materials is retarded by frictional resistance, somewhat the same as water flowing through a pipe. The intensity of the uplift can be controlled by construction of properly placed aprons, cutoffs, drains, and other devices.

Water pressures in the foundation can also initiate piping of weak zones within the foundation.

Therefore, exit gradients should be low enough to ensure that piping does not occur.

(b) Criteria.-For preliminary design purposes, uplift pressure distribution within a gravity dam, within its foundation, and at their contact are assumed to have an intensity at the line of drains equal to the tail water pressure plus one-third the differential between headwater and tail water pressures.

The pressure gradient is then extended linearly to headwater and tail water levels. If there is no tail water, a similar pressure diagram is determined using zero instead of the tail water pressure.

In all cases, pore pressures are assumed to act over 100 percent of the area.

For the final design, determination of the internal pressures within the dam should be based on the location and spacing of drains. Pressures in the foundation rock or at its contact with the dam should be determined based on geologic structures in the rock and on the location, depth, and spacing of drains. Flow nets computed by electric analog analysis, finite element analysis, or other comparable means should be used for the final determination of water pressure distribution.

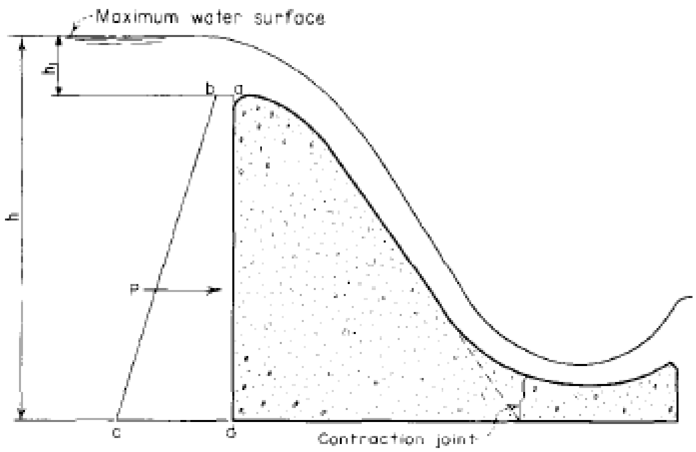


Figure 8-3.—Water pressures acting on an overflow concrete dam. 288-D-2506.

8.11. Dead Load.-(a) Basic Considerations.- The weight of the structure includes the weight of the concrete plus appurtenances such as gates and bridges. The total weight acts vertically through the center of gravity of the cross section, without transfer of shear between adjacent blocks.

(b) Criteria.-Total dead load is the weight of the concrete gravity structure plus the weight of appurtenances.

8.12. Ice.-(a) Basic Considerations.-Ice pressures can produce a significant load against the face of a dam in locations where winter temperatures are cold enough to cause relatively thick ice cover. Ice pressure is created by thermal expansion of the ice and by wind drag. Pressures caused by thermal expansion of the ice depend on the temperature rise of the ice, thickness of the ice sheet, the coefficient of thermal expansion, the elastic modulus, and the strength of the ice. Wind drag depends on the size and shape of the exposed area, the roughness of the surface, and the direction and velocity of the wind. Ice pressure is generally considered to be a transitory loading. Many dams are subjected to little, if any, ice pressure. The designer should decide, after consideration of the above factors, whether an allowance for ice pressure is appropriate.

(b) Criteria-The method of Monfore and Taylor [3] may be used to analyze anticipated ice pressures if the necessary basic data are available. When basic data are not available to compute pressures, an acceptable estimate of the ice load to be expected on the face of a structure may be taken as 10,000 lb/lin ft of contact between the ice and the dam for an assumed ice depth of 2 feet or more.

8.13. Silt Pressure.-(a) Basic Considerations. - During both normal flows and flood flows, silt eventually finds its way to the reservoir and is deposited in the still water adjacent to the dam.

Methods for determining the amount of silt and its deposition in a reservoir are discussed in appendix A. If allowed to accumulate against the upstream face of the dam, the saturated silt will exert loads greater than the hydrostatic pressure of water alone.

Sluiceways are often provided in gravity dams to reduce the accumulation of silt near the upstream face of the dam. In diversion dams, the main function of the sluiceway is to keep the head works and canal free from silt, thus reducing somewhat the silt load on the dam.

(b) Criteria.-In the absence of reliable test data, assume the saturated silt pressure is analogous and equivalent to that of a fluid having an 85 lb/ft3 horizontal component and a 120-lb/ft3 vertical component.

8.14. Earthquake.-(a) Basic Considerations.-Most earthquakes are the result of crustal movements of the earth along faults. Geologic examinations of the area should be made to locate all faults and to determine how recent the activity has occurred. Records of seismologic activity in the area should also be studied to determine the magnitude and location of all recorded earthquakes that may have affected the site.

When establishing earthquake events to be applied to structures, three levels of earthquake loading and response conditions should be considered: the OBE (operating basis earthquake), the DBE (design basis earthquake), and the MCE (maximum credible earthquake). The structural response condition expectations associated with each of these earthquakes are:

- OBE.-Structures, systems, and components necessary to the function of a project should be designed to remain operable under the vibratory ground motion of the OBE.
- DBE.-Under loading from the design basis event, the project should be designed to sustain the earthquake with reparable damage; however, those structures, systems, and components important to safety should remain operable. The degree of damage

acceptable would be based on an economic analysis or the estimated repair cost versus the initial cost to control the damage.

- MCE.-The structures of a project vital to retention or release of the reservoir would be designed for the loading from the MCE and would be required to function without permitting either a sudden uncontrolled release of the reservoir or a compromise in the controlled evacuation of the reservoir.

To determine the total forces caused by an earthquake, it is necessary to establish the earthquake magnitude and the distance from the site to the causative fault. Small or distant events usually produce little site and structural response. The curve shown on figure 8-4 suggests the need for analyses based on Richter magnitude and distance from causative fault. Bureau studies indicate that the fundamental period of vibration of a 50-foot-high section with a lo-foot-wide roadway varies from 0.086 to 0.05 second for downstream slopes of 0.5:l.O to 0.8:1.0, respectively. Dams less than 50 feet high have proportionately smaller fundamental periods. The second period of vibration is less than 0.02 second for all cases. For low dams, resonance is not likely to occur during earthquakes. Therefore, uniform accelerations from base to crest may be assumed; they are equal to the estimated site accelerations determined from figure 8-5. Assume the vertical acceleration is 50 percent of the horizontal acceleration. These accelerations can be used to compute inertia loads for pseudo-static analyses. In pseudo-static analyses, both vertical and horizontal earthquake loads should be applied in the direction that produces the least stable structure.

For the full reservoir condition, this will be a foundation movement in the upstream direction and a foundation movement downward. The upstream movement increases the downstream force of the water and silt loads and produces a downstream inertial force from the mass concrete in the dam.

The downward movement decreases the effective weight of the water above a sloping face and of the concrete in the dam. Both increasing the horizontal loads in a downstream direction and decreasing the effective weights tend to decrease the stability of the structure. The internal friction of silt may provide considerable damping as has been suggested in the literature [4]. However, until more exact data are determined, it is assumed that the dynamic effect of saturated silt is equivalent to that of water only.

Special conditions that warrant more detailed dynamic analyses are:

- Active faults directly beneath small dams
- Unusual geometry on small dams, such as large openings for waterways, bridge piers, etc.
- Large masses near the top of small dams, such as gates, bridges, etc.
- Dams higher than about 50 feet

More sophisticated methods of dynamic analysis are described in “Concrete Dams Design Standards,” which is currently (1986) being prepared by the Bureau.

The hydrodynamic pressures exerted on the dam face during earthquakes should be included in the analyses. For large gravity dams, the Bureau currently uses the method employed by [4] to incorporate the effects of hydrodynamic and foundation interaction.

In 1952, Zanger [5] presented formulas for computing the hydrodynamic pressures exerted on vertical and sloping faces by horizontal earthquake effects. These formulas derived by electric analogy and based on the assumption that water is incompressible, are applicable to very stiff small concrete gravity dams. For low dams, the error involved in computing the earthquake force on the water because of this simplifying assumption is probably less than 1 percent.

The effect of horizontal inertia on the concrete should be applied at the center of gravity of the mass, regardless of the shape of the cross section. For dams with vertical or sloping upstream faces, the increase in water pressure, P_e in pounds per square foot at any elevation due to horizontal earthquake,

is given by the following equation:

$$P_e = C\lambda w h \qquad (1)$$

where:

C = a dimensionless coefficient giving the distribution and magnitude of pressures,

$$C = \frac{C_m}{2}\left[\frac{y}{h}\left(2-\frac{y}{h}\right) + \left(\frac{y}{h}\left(2-\frac{y}{h}\right)\right)^{1/2}\right] \qquad (2)$$

λ = earthquake intensity = earthquake acceleration divided by acceleration due to gravity,

w = unit weight of water, in pounds per cubic foot,

h = total depth of reservoir at section being studied, in feet,

y = vertical distance from the reservoir surface to the elevation in question, in feet, and

Cm = maximum value of C for a given constant slope.

Values of C for various degrees of slope and relations of y and h may be obtained from figure 8-6.

The total horizontal force, V_e above any elevation y distance below the reservoir surface that is due to P, and the total overturning moment, M_e above that elevation due to P_e are:

$$V_e = 0.726\ P_e y \qquad (3)$$

and

$$M_e = 0.299\ P_e y^2 \qquad (4)$$

For dams with a combination vertical and sloping face, the procedure to be used is governed by the relation of the height of the vertical portion to the total height of the dam as follows:

- If the height of the vertical position of the upstream face of the dam is equal to or greater than one-half of the total height of the dam, analyze as if vertical throughout.
- If the height of the vertical portion of the upstream face of the dam is less than one-half of the total height of the dam, use the pressures on a sloping line connecting the point of intersection of the upstream face of the dam and reservoir surface with the point of intersection of the upstream face of the dam and the foundation.

On sloping faces of dams, the weight of the water above the slope should be modified by the appropriate acceleration factor [5]. The weight of the concrete also should be modified by this acceleration factor.

(b) Criteria.-The criteria used for establishing earthquakes events pertinent to the design should be as follows:

- OBE.-An earthquake that could be expected to occur once in 25year intervals during the economic life of the structure. The recurrence interval for this earthquake at the specific project would be established by the appropriate seismotectonic group. It is anticipated that this earthquake would be provided only for sites near highly seismically active areas for which the necessary information for developing recurrence interval relationships would be available.
- DBE.-An earthquake that would be likely to occur once in 200 years during the economic life of the structure. The recurrence interval for this earthquake for the project site would be set by the appropriate responsible group. The magnitude of this event is determined for each applicable area from recurrence interval relationships if an adequate amount of

seismic history data exists but, if not, the magnitude is estimated considering the geology and seismology of the area.

- MCE.-This earthquake would produce the most severe vibratory ground motion capable of being produced at the project site under the known tectonic framework. It is a rational and believable event that is in accord with all known geologic and seismologic facts. In determining the MCE, little regard is given to its probability of occurrence.

Methods of determining the above earthquakes representing the OBE, DBE, or MCE events should consider (1) historical records to obtain frequency of occurrence versus magnitude, (2) useful life of the structure, and (3) a statistical approach to determine probable occurrence of earthquakes of different magnitudes during the life of the structure.

When future developments produce such methods, suitable safety factors will be included in the criteria.

Reservoir-induced earthquakes should be considered in the analysis of a structure and its foundation when the reservoir area has parameters conducive to such an event. A dam and its associated foundation that could be affected by a reservoir-induced event should be designed for both a DBRIE (design basis reservoir-induced earthquake) and an ERIE (extreme reservoir-induced earthquake). The magnitude and location of these events should be based on tectonic, seismologic, and geologic site conditions and should be influenced by worldwide data on reservoir-induced seismicity. A reservoir-induced earthquake should be assumed to occur only on an active fault in the hydraulic regime of the reservoir. The DBRIE and the ERIE should have the same general level of probability of occurrence as the tectonic DBE and the MCE, respectively.

Criteria for level of damages, reparability, and safety factor for the dam and foundation should be the same for the DBE and DBRIE and for the MCE and the ERIE.

8.15. Load Combinations.-(a) Basic Considerations. - Gravity dams should be designed for all appropriate load combinations, using the proper safety factor for each. Combinations of transitory 326 DESIGN OF SMALL DAMS loads, each of which has only a remote probability of occurrence at any given time, have less probability of simultaneous occurrence and should not be considered as appropriate load combinations. For example, an expanding ice sheet is not a factor during a maximum flood, and the chances of an earthquake and a maximum flood occurring at the same time are extremely remote.

(b) Criteria-Gravity dams should be designed for the following load combinations using the corresponding safety factors.

(1) Usual load combinations.-Normal design reservoir elevation with appropriate dead loads, uplift, silt, ice, and tail water. If temperature loads are applicable to the specific sites, use minimum usual temperatures occurring at that time [a].

(2) Unusual load combinations.-Maximum design reservoir elevation with appropriate dead loads, silt, tail water, uplift, and minimum usual temperatures occurring at that time, if applicable.

(3) Extreme load combinations.-The usual loading plus the effects of the MCE.

(4) Other loads and investigations:

- The usual or unusual load combination with drains inoperative
- Dead load
- Any other load combination that, the designer thinks should be analyzed for a particular dam

D. FOUNDATION CONSIDERATIONS

The following paragraphs address, in general, foundation considerations associated with concrete dams. "Concrete Dam Design Standards" will address, in more detail, such topics as foundation deformation assessment, foundation seepage analysis, foundation sliding stability analysis, and foundation treatment.

8.16. Deformation Modulus.-(a) Basic Consideration- The deformation modulus is defined as the ratio of applied stress to elastic strain plus inelastic strain. It should be determined for each foundation material. Foundation deformations caused by loads from the dam affect the stress distributions within the dam. Conversely, response of the dam to external loads and foundation deformations determines the stresses within the foundation.

Proper evaluation of the dam-foundation interaction requires as accurate a determination of foundation deformation characteristics as possible at enough locations to make the evaluations meaningful.

Usually, the differential deformations are of concern, not the absolute magnitude of the deformation.

Foundation investigations should provide information related to or giving deformation moduli. The in situ modulus is usually determined by relationships involving laboratory tests on drill core specimens and fracturing characteristics, or by in situ jacking tests [6, 7, 8]. The in situ modulus should be determined for each material or for zones of similar material with different fracturing characteristics composing the foundation, including any fault or shear zone material. Fracturing in the rock mass reduces the in situ modulus to a value smaller than that measured on an intact core. Therefore, field data concerning rock mass fracture characteristics are helpful for approximating the in situ modulus.

Information on the variation of materials and their prevalence at different locations along the foundation is provided by drill hole logs, by tunnels in the foundation, by onsite inspections, and by good interpretive geologic maps, cross sections, and contour maps. Good compositional descriptions of the zone tested for deformation modulus and adequate geologic mapping and logging of the drill cores usually permit extrapolation of test results to untested zones of similar material.

(b) Criteria.-The following data relating to foundation deformability should be obtained for the analysis of a gravity dam:

- The effects of joints, shears, and faults obtained by direct (testing) or indirect (reduction factor) methods
- The deformation modulus of each type of material within and around the loaded area of the foundation

8.17. Shear Strength.-(a) Basic Considerations.- Resistance to shear within the foundation and between the dam and its foundation depends upon the shear strength inherent in the foundation materials and in the bond between the concrete rock contact. Shear strength properties can be determined from laboratory and in situ tests, field examination, and back calculation of slides. Evaluating shear strength properties of joints, joint infilling, faults, shears, seams, bedding, foliation, and of other adverse geologic structures should be included.

Assuming linearity is usually realistic for the shear resistance of intact rock over the range of normal stresses of interest. A curve of shear resistance versus normal stress is usually more realistic for open, rough discontinuities. However, it may be approximated by a linear relationship over the normal stress range of interest to the problem.

Smooth, open discontinuities usually exhibit linear behavior. The shear resistance versus normal stress relationship shown on figure 8-7 is determined from a number of tests at different normal stresses. The individual tests also give the relationship of shear resistance to displacement for a particular normal stress. The displacement used to determine the shear resistance is the maximum displacement that can be allowed on the possible sliding plane without causing unacceptable stress concentrations within the dam or foundation.

Because specimens tested in the laboratory or in situ are small compared with the foundation, the scale effect should be carefully considered in determining the values of shear resistance to be used.

Items to consider when assessing large scale behavior should include joint characteristics, fracturing, and variability within similar rock types.

When a foundation is nonhomogeneous, the possible sliding plane may consist of several different materials, some intact and some fractured. Intact rock reaches its maximum break bond resistance with less deformation than is necessary for fractured materials to develop their maximum frictional resistances. Therefore, the shear resistance developed by each fractured material depends upon the displacement of the intact rock part of the potential sliding plane considered in the analysis. An adequate number of tests, as determined by the designer, should be made to obtain a shear resistance versus normal load relationship for each material along the possible sliding planes. The value of shear resistance recorded during tests should include measurements at normal stress levels that correspond to those expected to occur in situ. The total shear resistance against potential sliding along nonhomogeneous foundation planes is the summation of the shear resistance of all the materials along the plane, at compatible shear displacements.

For the shear strength of soil-like foundation materials, many static shear tests have been made and the results published. However, published results should only be used as a guide. For use in design, the shear strength characteristics of the site specific foundation materials should be determined by testing.

(b) Criteria.-Foundation shear strength properties can be determined from laboratory and in situ tests and, in some cases, by field examination and back calculation of slides. The shear strength should be determined for joints, joint infilling, faults, shears, seams, bedding, foliation, and any other geologic feature that may influence stability.

Scale effects should be carefully considered when applying shear strength properties obtained from test specimens.

8.18 Foundation Configuration.-(a) Basic Considerations.-The thickness of a gravity dam at the contact with the foundation and the slope of the concrete-rock contact are factors important to the stability of the structure. Transversely, the foundation contact should be either horizontal or, preferably, sloping upstream. The transverse thickness is usually determined by the dimension necessary for the structure to satisfy stress and stability requirements. Longitudinally, the profile should vary smoothly without abrupt changes to minimize stress concentrations.

(b) Criteria.-The foundation contact in the upstream-downstream direction should be either horizontal or sloping upstream. In addition, the foundation contact should vary smoothly, without any abrupt changes, along the profile of the dam.

E. REQUIREMENTS FOR STABILITY

8.19. Safety Factors.-(a) Basic Considerations.- All loads used in design should be chosen to represent, as nearly as can be determined, the actual loads that will occur on the structure during operation, in accordance with the criteria under "Load Combinations" (sec. 8.15). Methods of determining the load-resisting capacity of the dam should be the most accurate available. All uncertainties regarding loads or load-carrying capacity should be resolved as far as practicable by field or laboratory tests and by thorough exploration and inspection of the foundation. Thus, the safety factor should be as accurate an evaluation as possible of the ability of the structure to resist applied loads.

Although somewhat lower safety factors may be permitted for limited local areas within the foundation, overall safety factors for the dam and its foundation, including the contributions from any remedial treatment, should meet requirements for the load combination analyzed.

For other load combinations where safety factors are not specified, the designer is responsible for selecting safety factors consistent with those for the load combination categories previously discussed (sec. 8.15(b)). Somewhat higher safety factors should be used for foundation studies because of the greater amount of uncertainty involved in assessing foundation load-resisting capability.

Safety factors for gravity dams are based on the use of the gravity method of analysis, and those for foundation sliding stability are based on an assumption of uniform stress distribution on the plane being analyzed.

Like other important structures, dams should be regularly and frequently inspected. Adequate observations and measurements should be made of the structural behavior of the dam and its foundation to ensure that the structure is functioning as designed.

A concrete gravity dam must be designed to resist, with ample safety factor, internal stresses and sliding failure within the dam and foundation. The following subsection discusses recommended allowable stresses and safety factors.

(b) Criteria.-(l) Compressive Stress.-The maximum allowable compressive stress for concrete in a gravity dam subjected to any of the usual load combinations should not be greater than the specified compressive strength divided by a safety factor of 3.0. Under no circumstances should the allowable compressive stress for the usual load combinations exceed 1,500 lb/in2.

A safety factor of 2.0 should be used in determining the allowable compressive stress for the unusual load combinations. The maximum allowable compressive stress for the unusual load combinations should never exceed 2,250 lb/in’.

The maximum allowable compressive stress for the extreme load combinations should be determined in the same way using a safety factor greater than 1.0.

Safety factors of 4.0, 2.7, and 1.3 should be used in determining allowable compressive stresses in the foundation for usual, unusual, and extreme load combinations, respectively.

(2) Tensile Stress.-In order not to exceed the allowable tensile stress, the minimum allowable compressive stress computed without internal water pressure should be determined from the following expression, which takes into account the tensile strength of the concrete at the lift surfaces:

$$\sigma_{z_u} = pwh - \left(\frac{f_t}{s}\right) \qquad (5)$$

where:

σ_{zu}= minimum allowable stress at the face,

p = reduction factor to account for drains,

w = unit weight of water,

h = depth below water surface,

f_t = tensile strength of concrete at lift surfaces, and

s = safety factor.

All parameters must be specified using consistent units.

The value of p should be 1.0 if drains are not present, inoperable, or if cracking occurs at the downstream face and p should be 0.4 if drains are used. The value 0.4 represents the approximate stress at the upstream face caused by uplift pressures within the dam, assuming drains are spaced 5 percent of the reservoir depth from the upstream face, no tail water level is included, and drains are fully operable. A more accurate determination of p is required if drains are spaced farther from the face, if tail water is included, or if the drains are operating at less than 100 percent efficiency. A safety factor of 3.0 should be used for usual, 2.0 for unusual, and 1.0 for extreme load combinations.

The allowable value of σ_{zu}, for usual load combinations should never be less than 0. Cracking should be assumed to occur if the stress at the upstream face is less than σ_{zu}. Cracking is not allowed for the usual and unusual load combinations for new dams; however, cracking is permissible for the extreme load combination if stability is maintained and allowable stresses are not exceeded (see sec. 8.22).

(3) Sliding Stability. - The shear-friction safety factor provides a measure of the safety against sliding or shearing on any section. The following expression is the ratio of resisting to driving forces and applies to any section in the structure, in the foundation, or at its contact with the foundation for the computation of the shear-friction safety factor,

$$Q = \frac{CA + (\Sigma N + \Sigma U) \tan \phi}{\Sigma V} \qquad (6)$$

where:

Q = unit cohesion,

A = area of section considered,

$\sum N$ = summation of normal forces,

$\sum U$ = summation of uplift forces,

tan ϕ = coefficient of internal friction, and

$\sum V$ = summation of shear forces.

All parameters must be specified using consistent units. Uplift is negative according to the sign convention in [11].

The minimum shear-friction safety factor within the dam or at the concrete-rock contact should be 3.0 for usual, 2.0 for unusual, and greater than 1.0 for the extreme load combinations. The safety factor against sliding on any plane of weakness within the foundation should be at least 4.0 for the usual, 2.7 for unusual, and 1.3 for the extreme load combinations [l]. If the computed safety factor is less than required, foundation treatment can be included to increase the safety factor to the required value. For concrete structures on soil-like foundation materials, it is usually not feasible to obtain safety factors equivalent to those prescribed for structures on competent rock. Therefore, safety factors for concrete dams on non-rock foundations are left to the engineering judgment of an experienced designer.

F. STRESS AND STABILITY ANALYSES

The following paragraphs address, in general, considerations relating to sliding stability and internal stresses. The "Concrete Dam Design Standards" will also address these subjects. Additional details are also contained in [11].

8.20. Sliding Stability.-(a) Basic Considerations.-The horizontal force, ZV on figure 8-l (A), tends to displace the dam in a horizontal direction.

This tendency is resisted by the shear resistance of the concrete or the foundation. As previously discussed in this chapter, the shear strength characteristics of both the concrete and the foundation should be determined by testing.

For sliding within the foundation, the orientation of joints, faults, and shears should be investigated to help identify rock blocks and potential modes of instability. Attention should also be given to joint continuity to help assess the potential for instability.

The rigid block method of analysis, which assumes a uniform stress distribution on the potential sliding plane analyzed, should be sufficient for most cases. However, for cases where rigid block analysis may not be applicable, such as cases involving a variable foundation deformation modulus or special cases involving foundation treatment, finite element modeling may be warranted to more accurately predict stress levels and distributions.

For situations where the recommended safety factors for sliding stability are not satisfied, several remedies are available.

For unsatisfactory stability within the dam, reshaping the dam, increasing the strength of the concrete, and installing posttensioned cables are some possible solutions. Site-specific feasibility and cost effectiveness are factors to consider when selecting the proper alternative.

Unsatisfactory safety factors are more common within the foundation. Various methods of foundation treatment can improve sliding stability.

Drainage can reduce uplift forces. Posttensioned cables and rock bolts can increase the normal force acting on a potential sliding plane. Concrete shear keys are also an effective method of foundation treatment. Potential sliding surfaces in the foundation can be intercepted by a key trench excavation.

Backfilling the key trench with mass concrete allows the shear strength of the key to be incorporated in the sliding analysis.

Concrete cutoff walls are often provided on structures constructed on soil-like foundations. A properly located and designed cutoff wall engages an additional volume of foundation materials that must be moved before the structure can slide. Sliding stability should also be investigated along any weaker stratum that may exist at depths below the bottom of the cutoff wall.

(b) Criteria.-The rigid block method of analysis should be sufficient for most cases. However, the finite element method should be used for cases that are not expected to have a uniform stress distribution along the potential failure surface.

To assess foundation sliding stability, the orientation and continuity of joints, faults, and shears should be investigated to help identify rock blocks and potential modes of instability.

8.2 1. Internal Stresses-Untracked Sections.-(a) Basic Considerations-For most gravity dams, internal stresses can be adequately determined for a cross section using the gravity method of analysis. It is applicable to the general case of a gravity section with a vertical upstream face and with a constant downstream slope and to situations where there is a variable slope on either or both faces. The gravity method is substantially correct, except for horizontal planes near the base of the dam where foundation yielding is reflected in stress calculations. Therefore, where necessary in the judgment of an experienced design engineer, finite element modeling should be used to check stresses near the base of a dam. Other methods of analysis, such as the finite element method should also be used to analyze three-dimensional behavior [l]. Grouted or keyed contraction joints, and monolithically constructed RCC dams exhibit three-dimensional behavior, especially along changes in foundation grade or changes in foundation deformation modulus.

The gravity method of analysis uses the following formula to determine the stress distribution along a horizontal plane within the dam:

$$\sigma_z = \frac{\Sigma W}{A} \pm \frac{\Sigma My}{I} \qquad (7)$$

where:

σ_z, = normal stress on a horizontal plane,

∑W = resultant vertical force from forces above the horizontal plane,

A = area of horizontal plane considered,

∑M = summation of moments about the center of gravity of the horizontal plane,

y = distance from the neutral axis of the horizontal plane to where o, is desired, and

I = moment of inertia of the horizontal plane about its center of gravity.

Uplift from internal water pressures and stresses caused by the moment contribution from uplift along a horizontal plane are usually not included in the computation of σ_z. These stress contributions are considered separately as described in the tensile stress criteria (sec. 8.19(b) (2)).

(b) Criteria-Internal stresses can be computed by the gravity method of analysis to determine the stress distribution along a horizontal plane within the dam. The method may not be applicable near the base where foundation yielding may influence results or for three-dimensional behavior. The effects from uplift are not considered in the computation of stresses, but are considered separately in accordance with the tensile stress criteria (sec. 8.19(b) (2)).

8.22. Internal Stresses and Sliding Stability - Cracked Sections.-(a) Basic Considerations.-

Applied loads tend to produce tension along the upstream face of concrete gravity dams. In general, when allowable concrete tensile strength is exceeded, a crack is assumed to form and propagate horizontally to the point of zero stress, leaving the remaining untracked section entirely in compression.

Cracking does not occur at all points where excessive tension is indicated, but usually only at the point of maximum tension on each face. However, if cracking at maximum tension location does not sufficiently relieve tension at the other locations, it may be necessary to assume cracking at additional points along the face.

New dams should be designed not to crack for all static loading combinations; however, cracking is permissible for earthquake loading if it can be shown that stress and stability criteria is satisfied during and after the earthquake event. It is also permitted for analyses to indicate that cracking is likely for existing dams, for the condition of maximum water surface with drains inoperative, as long as it can be shown that stress and stability criteria is satisfied.

For various reasons, cracking has occurred in many existing dams. The observed or suspected existence of a crack on either the upstream or downstream face does not necessarily signify instability; however, a crack warrants close examination and, especially, documentation to monitor enlargement or associated deterioration. Investigative methods include core drilling, sonic measurements, and inplace testing. Once the crack location and extent have been identified, stability analyses are essential to evaluate consequences from the various load combinations.

If analyses indicated that unacceptable cracking is likely to occur for new or existing dams, or show that an existing crack has reduced stability to unacceptable levels, modifications should be made to remedy the situation. Some possible modifications are increasing the thickness of the dam, installing post-tensioned cables, installing drains to reduce the uplift from internal water pressures, or increasing the concrete strength.

Cracking should be assumed to occur when analyses indicate the vertical normal stress at the face, computed in accordance with section 8.21, is less than the minimum required stress as computed by equation (5). Once cracking is indicated, a cracked section analysis is necessary. This involves estimating the potential penetration of a horizontal crack from the upstream face, and then computing the stress distribution and shear-friction safety factor along the untracked portion.

(b) Static Method of Analysis.-Assumptions associated with a static, cracked-section analysis are:

- Stress distribution along a horizontal section, without uplift, varies linearly between upstream and downstream faces.
- Once a crack occurs, uplift pressure equivalent to reservoir pressure above the crack exists throughout the entire crack depth. This is a conservative assumption because if drains exist they are considered inoperable or uneffective after cracking occurs. Uplift is then assumed to vary linearly from crack tip to tail water pressure at downstream face.
- Crack penetrates to point of zero stress. This assumes no tensile strength at crack tip, which means the entire untracked length is entirely in compression.

Based on these assumptions, the following equations have been developed to estimate crack length and the resulting stress at the downstream face. The equations apply to the general static case shown on figure 8-8.

$$e' = \frac{\Sigma M}{\Sigma W - \overline{A3} \cdot T} \qquad (8)$$

$$T_1 = 3\left(\frac{T}{2} - e'\right) \qquad (9)$$

$$\overline{B5} = \frac{2(\Sigma W - \overline{A3} \cdot T)}{T_1} + \overline{A3} \qquad (10)$$

where:

e’ = eccentricity of stress diagram after cracking, which is distance from resultant normal force on horizontal section to center of gravity of base at T/2;

∑M = summation of moments from all forces, ∑W and ∑V on figure 8-8(A), but excluding resultant and uplift forces that act on horizontal plane;

∑W = summation of vertical forces, excluding uplift and resultant force;

A3 = water pressure at upstream face; equivalent to full reservoir water pressure at elevation in question;

T = thickness of section;

T, = thickness of untracked segment; and

B5 = stress at downstream face.

These equations can be derived by examining figure 8-8 and by realizing that the weight (∑W) and the moment (∑M) are resisted by a combination of the resultant and uplift forces that act on the horizontal plane. On figure 8-8(D), the geometric shape defined by AB43 represents the uplift pressure diagram, and triangle B54 represents the pressure diagram that defines resultant force. For the purpose of deriving equations (8), (9), and (lo), consider only the geometry of the combined pressure diagram. Because the pressure distribution represented by the combined diagram is all directed upward, the diagram can be separated into rectangle ABB’3 and triangle B’54. Using statics and separating combined diagram in this manner, the summation of vertical forces produces:

$$\Sigma W - A_{CD} = 0 \qquad (11)$$

where:

A_{CD} = area of combined diagram

$$= \overline{A3} \cdot T + (\overline{B5} - \overline{A3}) \left(\frac{T_1}{2}\right) \qquad (12)$$

Solving equation (11) for B5 results in equation (10). Summation of moments about the center of gravity of the base produces:

$$\Sigma M + (\overline{A3} \cdot T)(0) - (\overline{B5} - \overline{A3}) \left(\frac{T_1}{2}\right)\left(\frac{T}{2} - \frac{T_1}{3}\right) = 0 \quad (13)$$

From figure 8-8(D), it can be seen that:

$$e' = \frac{T}{2} - \frac{T_1}{3} \qquad (14)$$

Rearranging equation (14) yields equation (9). In equation (13), substitute e' for (T/2) - (T_1/3), and for B5, substitute expression from equation (10). Solving resulting expression for e' produces equation (8).

(c) Pseudo-static Method of Analysis.-To perform a pseudo-static, cracked-section analysis, a similar set of equations to those in subsection (b) can be derived in a similar manner. However, there is one major difference in the 'assumptions associated with the earthquake, cracked-section analysis.

When a crack develops during an earthquake event, uplift pressure within the crack is assumed to be zero. This assumption is based on studies that show the opening of a crack during an earthquake event relieves internal water pressures, and the rapidly cycling nature of opening and closing the crack does not allow reservoir water, and the associated pressure, to penetrate. Based on this assumption and the other assumptions for the static, cracked section analyses, the following equations have been developed for pseudo-static, earthquake, cracked section analyses. These equations apply to the general case shown on figure 8-9.

$$e' = \frac{\Sigma M + M_u}{\Sigma W - \overline{A'4}(T_1)} \qquad (15)$$

$$\overline{B5} = \frac{2[\Sigma W - (\overline{A'4} \cdot T_1)]}{T_1} + \overline{A'4} \qquad (16)$$

where:

e’ = eccentricity of stress diagram after cracking, which is distance from resultant normal force on horizontal section to center of gravity of base at T/2;

∑M = summation of moments from all forces including earthquake forces, GW and CV on figure 8-9(A), but excluding the resultant and uplift force that act on the horizontal plane;

M_u = moment from rectangle A’BB’4 portion of combined pressure diagram on figure 8-9(D);

∑W = summation of vertical forces, excluding uplift and resultant force;

A’ 4 = uplift pressure at end of crack, see figure 8-9(C);

T_1 = thickness of untracked segment; and

B5 = stress at downstream face.

Using equations (15) and (16) to determine crack depth is an iterative process because the uplift pressure that remains in the untracked portion depends on the crack depth, and the crack depth depends partially on the remaining uplift. If cracking is indicated at upstream face, the pressure diagram should be revised as on figure 8-9(D). For an initial assumption, a crack depth equal to one-half the thickness can be used, Uplift effects in the uncracked section can then be determined using the uplift pressure diagram shown on figure 8-9(C).

This particular uplift diagram represents drained conditions and is discussed in section 8.10. Once the uplift effects are known, the depth of crack may be determined from equations (9) and (15). The computed crack depth is then compared to the estimated crack depth. If a satisfactory degree of accuracy has not been obtained, a new crack depth is estimated and the process repeated until satisfactory accuracy is obtained.

If stability and stress levels are satisfactory for the cracked section during the earthquake event, post-earthquake static conditions should also be checked. Post-earthquake analyses should include full uplift pressure throughout the crack.

(d) General Iterative Method of Analysis.-Instead of using the equations in subsections (b) and (c) for static and pseudo-static cracked-section analyses, an iterative method can be used that produces the same results. Using this method, variations of the conditions depicted on figures 8-8 and 8-9 can be readily incorporated. This iterative method also allows the mechanics of cracked-section analysis to be easily discernible, and furnishes a greater appreciation of the factors that influence crack propagation.

To begin this iterative method, crack initiation is still determined as previously explained. Basically, a crack is assumed to form when the vertical normal stress at the upstream face, computed in accordance with section 8.21, is less than the minimum required stress as computed by equation (5).

Once a crack is initiated, a crack depth is estimated and the center of gravity shifts to the center of the untracked portion. Moments about this center of gravity are computed and summed. Moment contributions from all forces, including uplift, are included in this summation of moments. Stress at the crack tip is then computed using equation (7) and should include uplift in the CW and CM terms of this equation. The moment of inertia for equation (7) is now based only on the untracked length.

Based on the computed stress at the crack tip, the estimated crack length is adjusted toward the point of zero stress. The process is t>hen repeated until the crack tip is at, the point of zero stress.

(e) Criteria.-Cracked-gravity sections require that stress and stability analyses account for the) effects from the crack. The analysis process involves determining the crack depth and resulting stress distribution across the untracked length.

Regardless of the method used to determine the depth of crack, stress and stability criteria need to be checked for the untracked portion. Equation (6) is used to compute sliding stability, but cohesion is considered only along the untracked length. Sliding stability and stress levels are considered satisfactory if the criteria established in section 8.19 are satisfied.

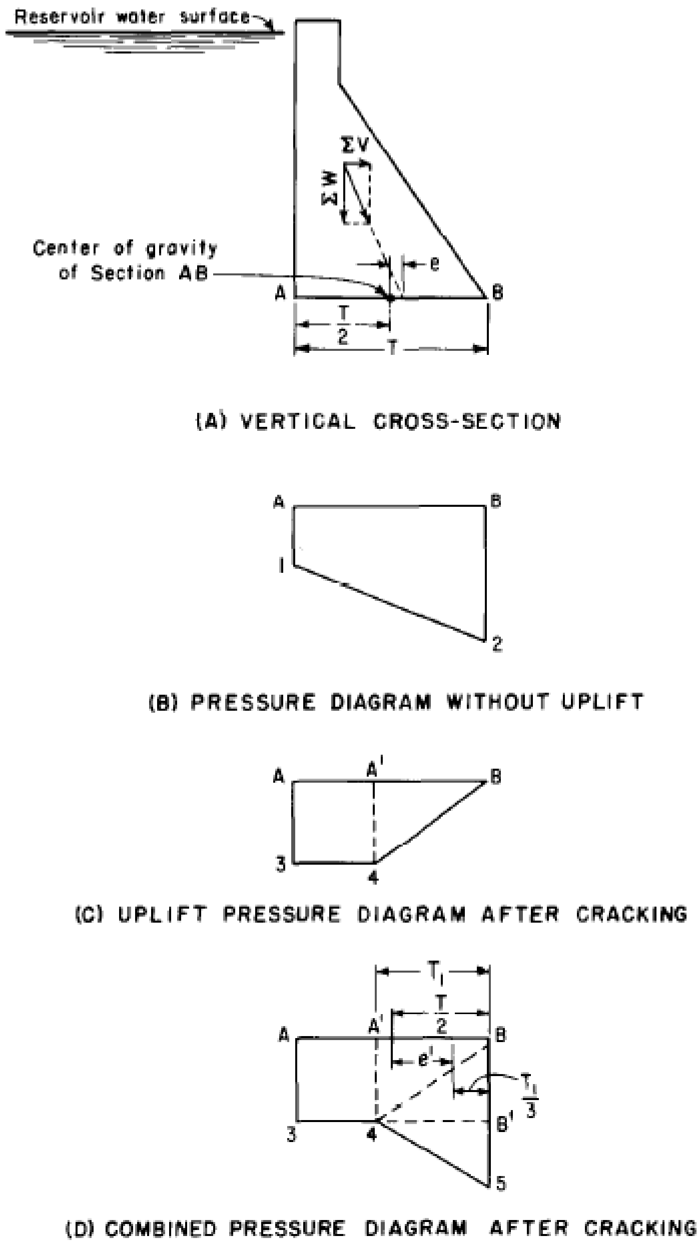


Figure 8-8.—Static pressure diagrams along the base or any horizontal section of a gravity dam. 103–D–1871.

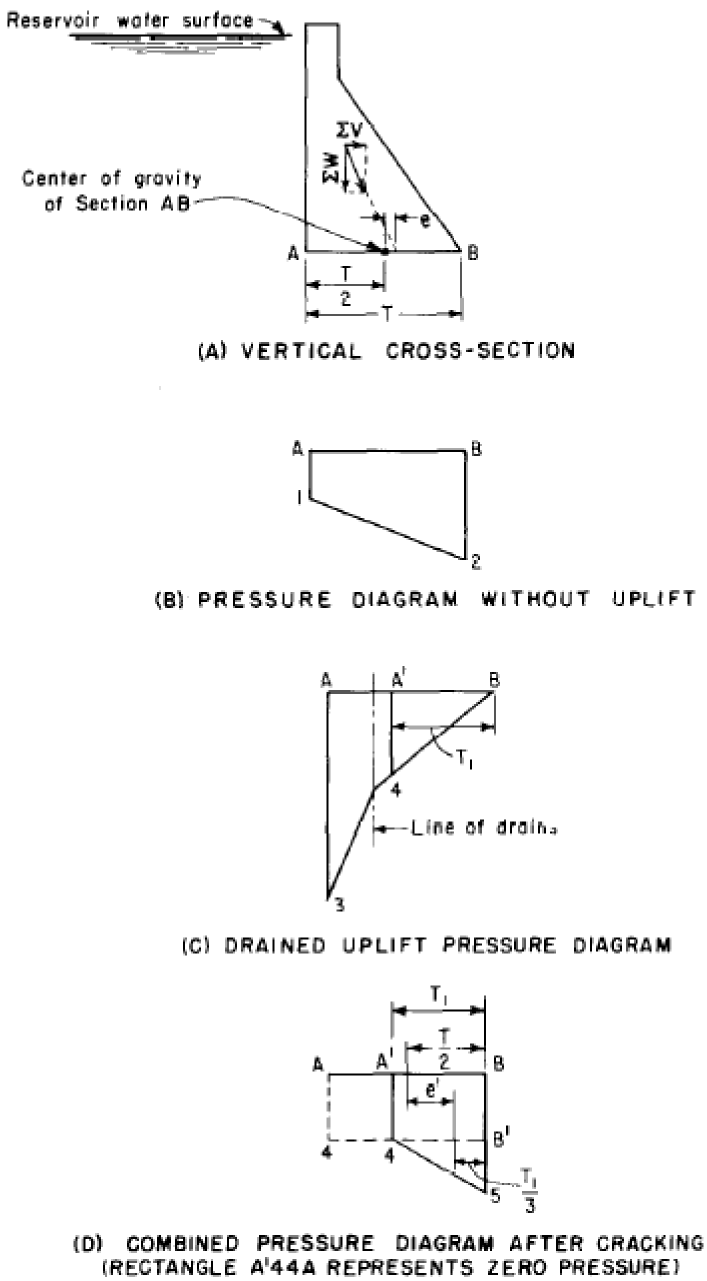


Figure 8-9.—Pseudostatic pressure diagrams along the base or any horizontal section of a gravity dam. 103-D-1872.

G. ADDITIONAL TOPICS

8.23. Dams on Pervious Foundations.-The design of dams on pervious (soil-like) foundations involve problems of erosion of the foundation material, settlement, and seepage under the structure.

The complexity of these problems varies greatly and depends on the type, stratification, permeability, homogeneity, and other properties of the foundation materials, as well as the size and physical requirements of the structure itself.

The design of concrete gravity storage dams and diversion dams more than 30 feet high on pervious foundations usually requires extensive field and laboratory investigations. Such structures are beyond the scope of this text, which for pervious foundations is limited to gravity dams whose maximum net head (headwater to tail water) is not appreciably greater than 20 feet.

The control of erosion, seepage, and uplift forces under dams constructed on pervious foundations often requires the use of some or all of the following devices:

- Upstream apron, usually with cutoffs at the upstream end
- Downstream apron, with scour cutoffs at the downstream end, and with or without filters and drains under the apron
- Cutoffs at, the upstream or downstream end or at both ends of the overflow section, with or without filters or drains under the sect ion

A concrete apron may be placed upstream of the dam in conjunction with one of the various types of cutoff walls. The function of the apron is to increase the length of the path of percolation to reduce uplift under the main portion of the dam.

Downstream concrete aprons have two functions; they lengthen the path of percolation in the foundations and provide a basin where the energy of the overflowing water can be safely dissipated. Energy dissipation on the concrete helps to prevent dangerous erosion at the toe of the dam. Where it is not feasible to construct a concrete apron long enough to completely avoid erosion, additional protection may be gained by placing riprap downstream from the apron.

Cutoff walls can be constructed under aprons or under the dam itself to prevent or reduce under effective cutoff walls; these include concrete walls, steel sheet piling, impervious earth compacted in a trench, and cement-bound curtains.

Cement-bound curtains (sec. 6.10(e)) are composed of overlapping columns consisting of a mixture of cement and the pervious foundation material. The columns are mixed in place and are formed by jet grouting or similar techniques.

A concrete cutoff is probably the best type of cutoff for preventing under seepage and is often used. In addition to acting as a cutoff, it can be designed to contribute substantially to the stability (sliding resistance) of the dam when placed under the dam structure.

Reduction of uplift pressure under the downstream apron or the downstream toe of the dam may be accomplished by pipe drains. Drains are often PVC (polyvinyl chloride) pipe laid in graded material, which acts as a filter. They may be perforated or plain pipe laid with open joints. The drains may be located at the downstream toe of the dam, at selected places under the downstream apron, and immediately upstream from the downstream cutoff.

Weep holes are commonly used for reduction of uplift pressure under aprons and excessive pressure behind walls. It is important that the gradation of the filter materials used in conjunction with the weep holes be carefully selected with respect to the gradation of the foundation materials to prevent piping, see section 6.10(i).

8.24. Details of Layout and Design.-

(a) Non-overflow Sections.-The elevation of the top of a non-overflow dam should be established by assuming a safe freeboard above the maximum high-water surface in the reservoir. The freeboard should be sufficient to allow for the maximum wave height, as given in table 6-7. Although only one-half of the wave height is above the mean water level, the full height is ordinarily used to allow for wave run up on the face of the dam. A minimum freeboard of 3 feet is recommended for most small concrete dams.

The top width is determined by such requirements as climatic conditions, and the need for travel across the dam and for access to the gate-operating mechanism. A top width of less than 4 feet is not recommended.

The width of the base and the slope of the downstream face should be determined by a stability analysis. The customary method is to assume a section with the downstream face sloped approximately 0.70: l.0 (horizontally to vertically) and intersecting a vertical upstream face at the top of the dam. The assumed section is then analyzed and modified as required by the analysis until it meets the stability requirements. If the dam is stable about its base and about any section where there is a break in the continuity of the slope of either the upstream or downstream face, the portion of the dam between any of these sections is stable and does not require analysis.

Abrupt changes of slope on either face of the dam can cause unacceptable stress concentrations and should be avoided whenever possible. The usual intersection near the crest, formed by the vertical and sloping downstream faces, has been replaced with a circular fillet tangent to each face. Nominal size fillets effectively reduce stress concentrations, especially during earthquakes. Similarly, minimizing the mass near the crest helps reduce the inertia effects.

(b) Overflow Sections.-In general, the method for determining the stability of overflow dams is the same for non-overflow dams; however, additional considerations contribute to the configuration of overflow sections. The shape of the crest, the profile of the downstream face, and details of the energy dissipating basin or bucket are discussed in chapter 9. It is customary to provide a longitudinal contraction joint at the downstream toe, as shown on figure 8-3, and then only that portion of the dam upstream of the joint is used in the stability computations.

In cases where the dissipating device extends only a short distance downstream from the toe and is fairly massive, the contraction joint may be omitted.

The structure downstream from the toe is then included in the stability analysis and is so reinforced that it and the gravity portion will act as a unit. Under certain conditions, an upstream apron connected by reinforcement to the upstream face of the dam may be the most economical arrangement that will ensure stability.

Overflow dams using control features on the crest introduce-an additional problem. The forces acting on these features may produce tension in the upper portion of the dam, which will require adequate reinforcement.

(c) Contraction Joints.-If a conventionally placed concrete dam is appreciably longer than 50 feet, it is necessary to divide the structure into blocks by providing transverse contraction joints. The spacing of the joints is determined by the capabilities of the concrete equipment to be used and considerations of volumetric changes and attendant cracking caused by shrinkage and temperature variations. The possibilities of detrimental cracking can be greatly reduced by the selection of the proper type of cement and by careful control of mixing and placing procedures (see app. F). For normal conditions, a SO-foot spacing of contraction joints in constructing concrete dams is usually sufficient.

Where foundation conditions are such that undesirable differential settlement or displacement between adjacent blocks can occur, shear keys should be formed in the contraction joints. These may be formed vertically, horizontally, or in a combination of both, depending on the direction of the expected displacement. Leakage through the contraction joints is controlled by imbedding water stops, usually made of PVC, across the joints.

Module II

Spillway: Types, Design principles of Ogee spillway, side channel spillway, Chute spillway, Syphon Spillway, shaft Spillway, Gates & Valves. Energy dissipaters and stilling basin design. Outlet works.

Spillways

A. GENERAL

9.1. Function.-Spillways are provided for storage and detention dams to release surplus water or floodwater that cannot be contained in the allotted storage space, and for diversion dams to bypass flows exceeding those turned into the diversion system.

Ordinarily, the excess is drawn from the top of the reservoir and conveyed through a constructed waterway back to the river or to some natural drainage channel. Figure 9-l shows a small spillway in operation.

The importance of a safe spillway cannot be overemphasized; many failures of dams have been caused by improperly designed spillways or by spillways of insufficient capacity. Ample capacity is of paramount importance for earthfill and rockfill dams, which are likely to be destroyed if overtopped; whereas, concrete dams may be able to withstand moderate overtopping. Usually, the increase in cost is not directly proportional to the increase in capacity.

The cost of a spillway having ample capacity is often only moderately higher than the cost of a spillway that is too small.

In addition to providing sufficient capacity, the spillway must be hydraulically and structurally adequate and must be located so that spillway discharges do not erode or undermine the downstream toe of the dam. The spillway's bounding surfaces must be erosion resistant to withstand the high scouring velocities created by the drop from the reservoir surface to the tailwater level. Usually, a device is required to dissipate the energy of the water at the bottom of the drop.

The frequency of spillway use should be determined by the runoff characteristics of the drainage basin, which includes the nature of its development.

Ordinary river flows are usually stored in the reservoir, diverted through headworks, or released through outlets; the spillway is not required to function.

However, spillway flows do occur during floods or periods of sustained high runoff when the capacities of the other facilities are exceeded. Where large reservoir storage is provided or large outlet or diversion capacity is available, the spillway will be used infrequently. But at diversion dams where storage space is limited and diversions are relatively small compared with normal river flows, the spillway will be used almost constantly.

9.2. Selection of Inflow Design Flood.-

(a) General Considerations.-Flooding in an unobstructed stream channel is considered a natural event for which no individual or group is responsible.

However, when obstructions are placed across the channel, the project sponsors must either ensure that hazards to downstream interests are not appreciably increased or assume responsibility for damages resulting from operation or failure of the structures. The loss of the facility and the loss of project services and revenues occasioned by a failure should also be considered.

If danger to the structures alone were involved, the sponsors of many projects would prefer to rely on the improbability of an extreme flood occurrence rather than to incur the expense necessary to ensure complete safety. However, when the hazards involve downstream interests, including property damage and the loss of human life, a conservative attitude is required in the selection of the IDF (inflow design flood). Consideration of potential damage should not be limited to conditions existing at the time of construction. Probable future development in the downstream flood plain, encroachment by farms and resorts, construction of roads and bridges, and other future developments should be evaluated in estimating damages and hazards to human life that would result from a dam failure.

Dams impounding large reservoirs on principal rivers with high runoff potential should unquestionably be considered to be in the high-hazard category.

For such developments, conservative design criteria should be selected because failure could involve the loss of life or damages of disastrous proportions.

Conversely, small dams built on isolated streams in rural areas where failure would neither jeopardize human life nor create damages beyond the sponsor's financial capabilities may be considered to be in a low-hazard category. For such developments, design criteria may be established on a much less conservative basis. There have been numerous instances, however, where the failure of a small dam with small storage capacity has resulted in the loss of life and heavy property damage.

"Most small dams require a reasonable conservatism in design, primarily because a failure must not present a serious hazard to human life.

(b) Inflow Design Flood Hydrographs.-Chapter 3 "Flood Hydrology Studies" discusses the determination of flood hydrographs that can be used as inflow design floods. The procedures presented provide for the development of probable maximum floods and of specific-frequency floods.

Determination of the PMF (probable maximum flood) is based on the probability of simultaneous occurrence of the maximum of the several elements or conditions that can contribute to the flood. Such a flood is the largest that reasonably can be expected and is ordinarily accepted as the inflow design flood for dams whose failure would increase the danger to human life.

For a minor structure with significant storage where it is permissible to anticipate failure within the useful life of the project, a flood in the range of a 1 in 50 chance to a 1 in 200 chance of being equalled or exceeded may be used as the IDF. A discussion of these floods and their determination is given in section 3.12. Estimates of floods of these magnitudes may also be required to establish the capacity of a service or principal spillway in those cases where an auxiliary spillway will serve to augment the principal spillway.

9.3. Relation of Surcharge Storage to Spillway Capacity.-Stream flow is normally represented in the form of a hydrograph, which charts the rate of flow (discharge) in relation to time. A typical hydrograph representing a storm runoff is shown on figure 9-2. The flow into a reservoir at any time and the momentary peak can be read from curve A. The area under this curve is the volume of the inflow because it represents the product of rate of flow and time.

Where no storage is impounded by a dam, the spillway must be large enough to pass the peak of the flood. Therefore, the peak rate of inflow is of primary interest, and the total volume of the flood is of lesser importance. However, where a relatively large storage capacity above normal reservoir level can be made available economically by raising the dam, a portion of the flood volume can be retained temporarily in reservoir surcharge space and the spillway capacity may be reduced considerably. If a dam could be made sufficiently high to provide storage space to impound the entire volume of the flood above normal storage level, no spillway other than an emergency type would be required, provided the outlet capacity could evacuate the surcharge storage fast enough to accommodate a recurring flood. In such cases, a meteorologic study may be warranted to determine the interval between floods. In these cases the maximum reservoir level would depend entirely on the volume of the flood and the rate of inflow would be of no concern. From a practical standpoint, however, relatively few sites permit complete storage of an inflow design flood by surcharge storage. Such sites are usually off-channel reservoirs; that is, reservoirs that are supplied by canals and that have small tributary drainage areas.

In many reservoir projects, economic considerations necessitate a design that uses the surcharge storage. Determining the most, economical combination of surcharge storage and spillway capacity requires flood routing studies and economic studies of the costs of spillway-dam combinations, subsequently described. However, in making these studies, consideration must be given to the minimum size spillway that must be provided for safety. The IDF hydrographs are determined by the methods given in chapter 3. In many locations it is possible to estimate upper limit or probable maximum floods resulting from several meteorologic combinations, severe rain, rain falling on a snow pack, and snow melt runoff alone. In these cases each type of PMF hydrograph must be developed by the hydrologic engineer to enable the designer to testy each against each alternative design. Such a test ensures that the design selected will enable the completed structure to satisfactorily accommodate the most, critical flood.

9.4. Flood Routing.-The accumulation of storage in a reservoir depends on the difference between the rates of inflow and outflow. For an interval of time At, this relationship can be expressed by the equation:

$$\Delta S = Q_i \Delta t - Q_o \Delta t \qquad (1)$$

where:

ΔS = storage accumulated during Δt,

Qi = average rat,e of inflow during Δt, and

Q, = average rate of outflow during Δt.

The rate of inflow versus time curve is represented by the IDF hydrograph; the rate of outflow is represented by the spillway discharge versus reservoir- elevation curve; and storage is shown by the reservoir storage versus reservoir-elevation curve.

For routing studies, the IDF hydrograph is not variable once the inflow design flood has been selected.

The reservoir storage capacity also is not variable for a given reservoir site, so far as routing studies are concerned. The spillway discharge curve is variable: it depends not only on the size and type of spillway, but also on the manner of operating the spillway (and the outlets in some instances) to regulate the outflow.

The quantity of water a spillway can discharge depends on the type of control device. For a simple overflow crest the flow varies with the head on the crest, and surcharge storage capacity increases with an increase in spillway discharge. For a gated spillway, however, outflow can be varied with respect to reservoir head by operation of the gates. For example, one assumption for an operating gate-controlled spillway might be that the gates will be regulated so that inflow and outflow are equal until the gates are wide open; another assumption might be to open the gates at a slower rate so that surcharge storage will accumulate before the gates are wide open.

Outflows need not necessarily be limited to discharges through the spillway, but may be supplemented by releases through the outlets. In all such cases the size, type, and method of operation of the spillway and outlets with reference to the storages or to the inflow must be predetermined to establish an outflow-elevation relationship.

If equations could be established for the IDF hydrograph curve, for the spillway discharge curve (as may be modified by operational procedures), and for the reservoir storage curve, a solution of flood routing could be made by mathematical integration.

However, simple equations cannot be written for the IDF hydrograph curve or for the reservoir storage curve; therefore, such a solution is not practical.

Many techniques of flood routing have been devised, each with its advantages and disadvantages.

These techniques vary from a strictly arithmetical integration method to an entirely graphical solution.

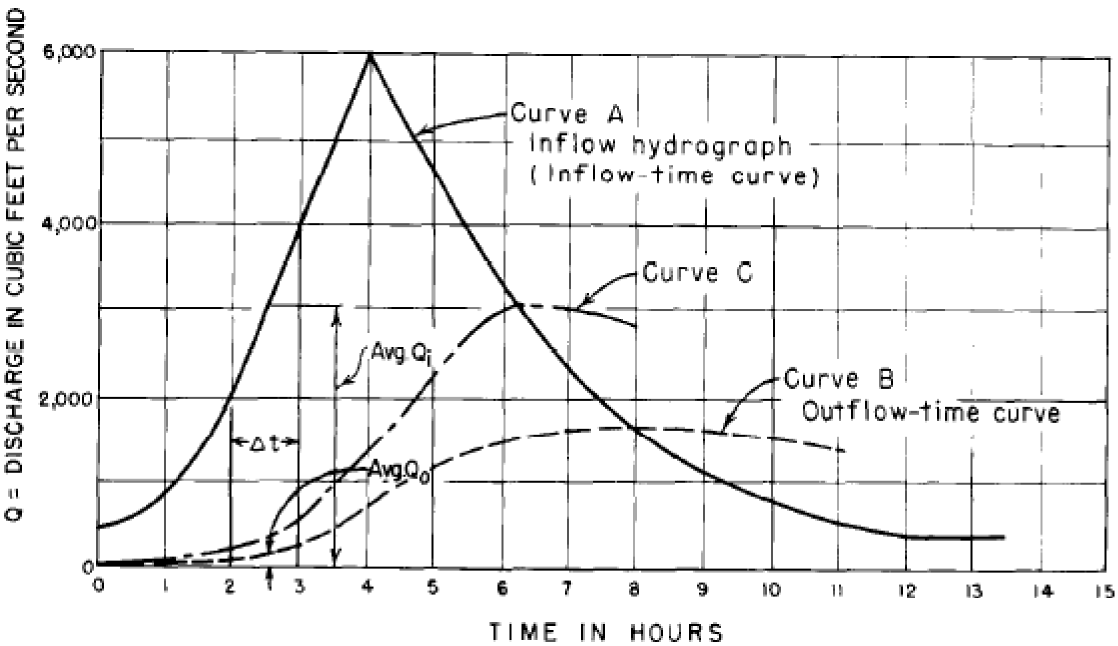


Figure 9-2.—Inflow and outflow hydrographs. 288–D–2399.

Computer programs have been developed and are generally available for use in flood routing. For simplicity, the arithmetical trial-and-error tabular method is illustrated in this text.

Table 9-l is an example of a flood routing for the data given on figures 9-2, 9-3, and 9-4. These data are necessary regardless of the method of flood routing used. They consist of the following:

- Inflow hydrograph (rate of inflow versus time), figure 9-2
- Reservoir capacity (reservoir storage versus reservoir elevation), figure 9-3
- Discharge curve (rate of outflow versus reservoir elevation), figure 9-4

The procedure for computations shown in table 9-l consists of the following steps:

1. Select a time interval, At, for column (2).
2. Obtain column (3) from the inflow hydrograph, figure 9-2.
3. Column (4) represents average inflow for At in cubic feet per second.
4. Obtain column (5) by converting column (4) values of second-feet for At to acre-feet (1 ft3/s for 12 hours = 1 acre-ft).
5. Assuming the trial reservoir water surface in column (6), determine the corresponding rate of outflow from figure 9-4, and record it in column (7).

6. Average the rate of outflow determined in step 5 above and the rate of outflow for the reservoir water surface that existed at the beginning of the period, and enter this average in column (8).

7. Obtain column (9) by converting column (8) values of cubic feet per second for At to acre-feet, similar to step 4 above.

8. Column (10) = column (5) - column (9).

9. The initial value in column (11) represents the reservoir storage at the beginning of the inflow design flood. Determine subsequent values for column (11) by adding AS values from column (10) to the previous column (11) value.

10. Determine reservoir elevation in column (12) corresponding to storage in column (11) from figure 9-3.

11. Compare reservoir elevation in column (12) with trial reservoir elevation in column (6). If they do not agree within 0.1 foot, make a second trial elevation and repeat procedure until such agreement is reached.

Table 9-1.—Flood routing computations.

(1) Time t, hours	(2) Δt, hours	(3) Inflow at time t, ft^3/s	(4) Average rate of inflow, Q_i for Δt, ft^3/s	(5) In-flow, acre-feet	(6) Trial reservoir storage-elevation at time t	(7) Outflow at time t, ft^3/s	(8) Average rate of outflow, Q, for Δt, ft^3/s	(9) Outflow, acre-feet	(10) Incremental storage ΔS, acre-feet	(11) Total storage, acre-feet	(12) Reservoir elevation at end of Δt, feet	(13) Remarks
0		400				0				1.050	300.3	
	1		600	50	~~300.2~~	~~5~~	~~3~~	~~0~~	~~50~~	~~1,100~~	~~300.3~~	~~High~~
1		800			300.3	10	5	0	50	1,100	300.3	OK
	1		1,400	117	~~300.8~~	~~84~~	~~32~~	~~3~~	~~114~~	~~1,214~~	~~301.0~~	~~High~~
2		2,000			301.0	80	45	4	113	1,213	301.0	OK
	1		3,000	250	~~302.3~~	~~300~~	~~190~~	~~16~~	~~234~~	~~1,447~~	~~302.1~~	~~Low~~
3		4,000			302.1	260	170	14	236	1,449	302.1	OK
	1		5,000	417	~~303.9~~	~~710~~	~~485~~	~~40~~	~~377~~	~~1,826~~	~~303.8~~	~~Low~~
4		6,000			303.8	690	475	40	377	1,826	303.8	OK
	1		5,350	446	~~305.6~~	~~1,060~~	~~675~~	~~73~~	~~373~~	~~2,199~~	~~305.3~~	~~High~~
5		4,700			305.3	1,160	925	77	369	2,195	305.3	OK
	1		4,000	333	~~306.3~~	~~1,500~~	~~1,900~~	~~111~~	~~222~~	~~2,417~~	~~306.2~~	~~Low~~
6		3,300			306.2	1,470	1,315	110	223	2,418	306.2	OK
	1		2,850	238	306.6	1,610	1,540	128	110	2,528	306.6	OK
7		2,400										
	1		2,000	167	306.7	1,650	1,630	136	31	2,559	306.7	OK
8		1,600										
	1		1,350	112	306.6	1,610	1,630	136	−24	2,535	306.6	OK
9		1,100										
	2		800	133	306.0	1,400	1,505	251	−118	2,417	306.1	High
11		500			306.1	1,430	1,520	253	−120	2,415	306.1	OK

The outflow time curve resulting from the flood routing shown in table 9-1 has been plotted as curve B on figure 9-2. As the area under the inflow hydrograph (curve A) indicates the volume of inflow, so the area under the outflow hydrograph (curve B) indicates the volume of outflow. It follows then that the volume indicated by the area between the two curves is the surcharge storage. The surcharge storage computed in table 9-l can, therefore, be checked by comparing it with the area measured on the graph.

A rough approximation of the relationship of spillway size to surcharge volume can be obtained without making an actual flood routing by arbitrarily assuming an approximate outflow-time curve and then measuring the area between it and the inflow hydrograph. For example, if the surcharge volume for the problem shown on figure

9-2 is sought where a 3,000-ft3/s spillway would be provided, an assumed outflow curve represented by curve C can be drawn, and the area between this curve and curve A can be measured by planimeter.

Curve C reaches its apex of 3,000-ft3/s where it crosses curve A. The volume represented by the area between the two curves indicates the approximate surcharge volume necessary for this capacity spillway.

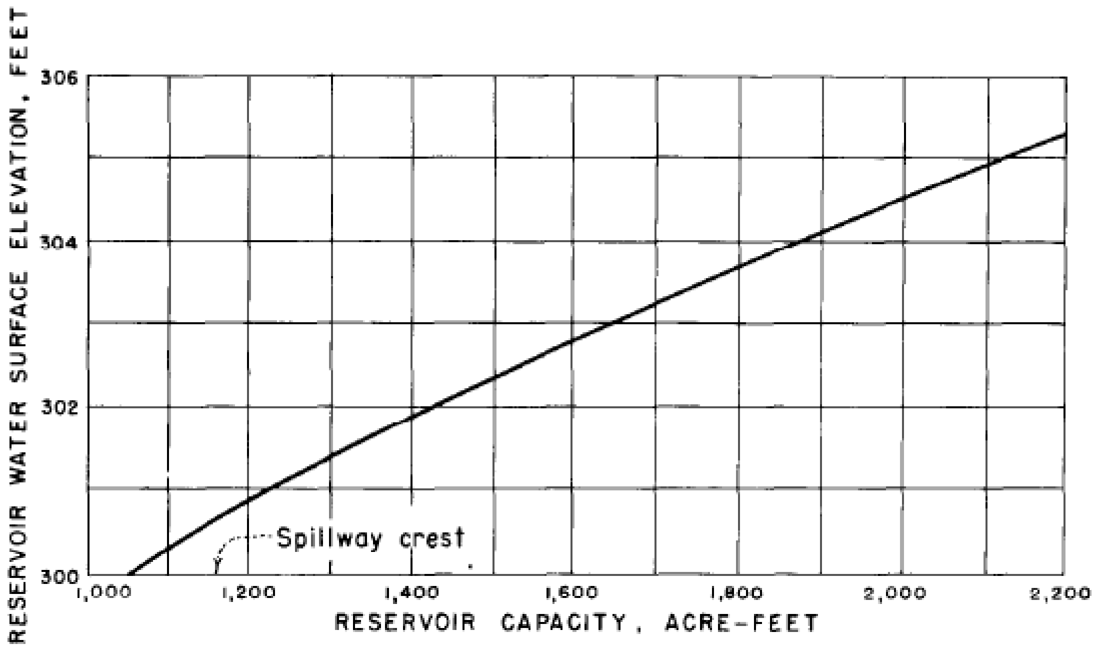
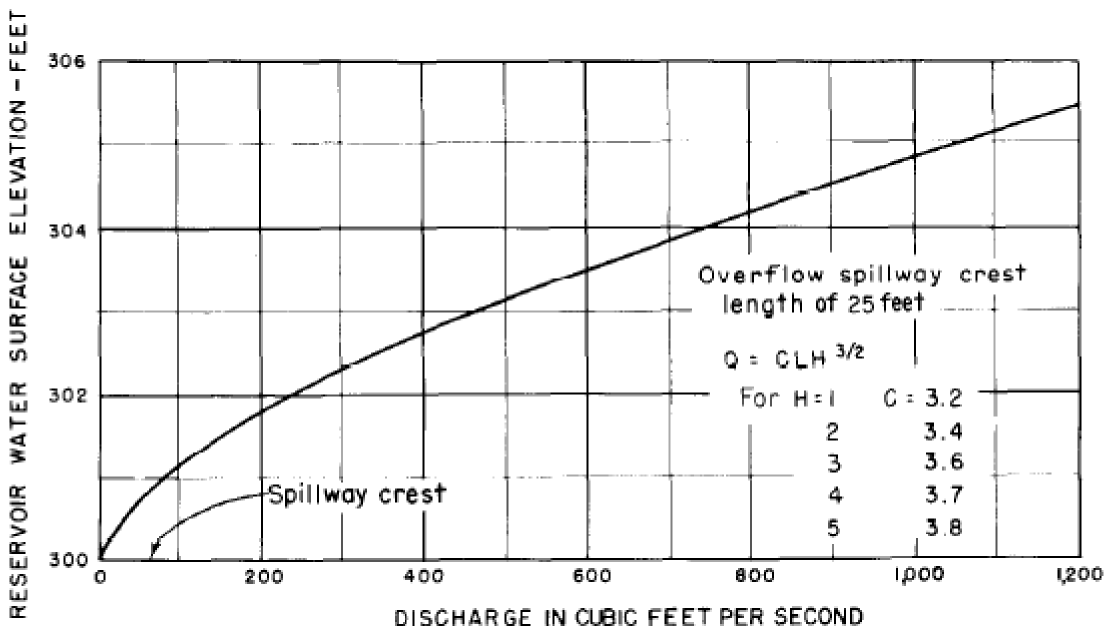


Figure 9-3.—Reservoir capacity curve. 288–D–2400.



Figuer 9-4.—Spillway discharge-elevation curve. 288–D–2401.

9.5. Selection of Spillway Size and Type.-

(a) General Considerations.-In determining the best combination of storage capacity and spillway capacity to accommodate the selected inflow design flood, all pertinent factors of hydrology, hydraulics, design, cost, and damage should be considered. In this connection and when applicable, consideration should be given to such factors as (1) the characteristics of the flood hydrograph, (2) the damages that would result if such a flood occurred without the dam, (3) the damages that would result if such a flood occurred with the dam in place, (4) the damages that would occur if the dam or spillway were breached, (5) the effects of various dam and spillway combinations on the probable damages upstream and downstream of the dam (as indicated by reservoir backwater curves and tail water curves), (6) the relative costs of increasing the capacity of spill ways, and (7) the use of combined outlet facilities to serve more than one function (e.g., control of releases and control or passage of floods.) Service outlet releases may be permitted in passing part of the inflow design flood unless such outlets are considered to be unavailable at the time of flooding.

The outflow characteristics of a spillway depend on the type of device selected to control the discharge.

These control facilities may take the form of an overflow weir, an orifice, or a pipe. Such devices may be unregulated, or they may be equipped with gates or valves to regulate the outflow.

After a spillway control device and its dimensions have been selected, the maximum spillway discharge and the maximum reservoir water level should be determined by flood routing. Other components of the spillway can then be proportioned to conform to the required capacity and to the specific site conditions, and a complete layout of the spillway can be established. Cost estimates of the spillway and dam should be made. Estimates of various combinations of spillway capacity and dam height for an assumed spillway type, and of alternative types of spillways, allow the selection of an economical spillway type and the optimum relationship of spillway capacity to height of dam. Figures 9-5 and 9-6 illustrate the results of such a study. The relationships of spillway capacities to maximum reservoir water surfaces obtained from the flood routings are shown on figure 9-5 for two spillways. Figure 9-6 illustrates the comparative costs for different combinations of spillway and dam, and indicates a combination that results in the least total cost.

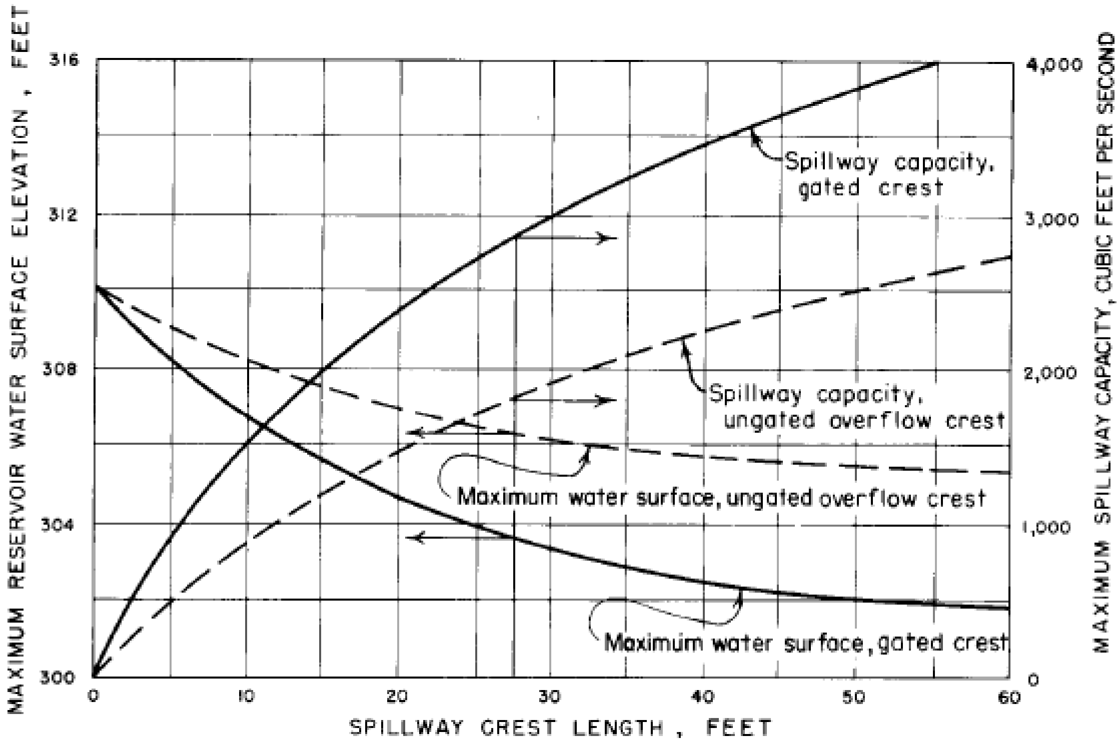


Figure 9-5.—Spillway capacity-surcharge relationship. 288–D–2402.

To make a study such as the one illustrated requires many flood routings, spillway layouts, and spillway and dam estimates. Even then, the study is not necessarily complete because many other spillway arrangements could be considered. However, a comprehensive study to determine alternative optimum combinations and minimum costs may be warranted for large dams, but not for the design of small dams. The designer's judgment is required to select for study only the combinations that show definite advantages, either in cost or adaptability. For example, although an ungated spillway might be slightly more expensive than a gated spillway, it may be more desirable because of its less complicated construction, its automatic and trouble-free operations, its ability to function without an attendant, and its less costly maintenance.

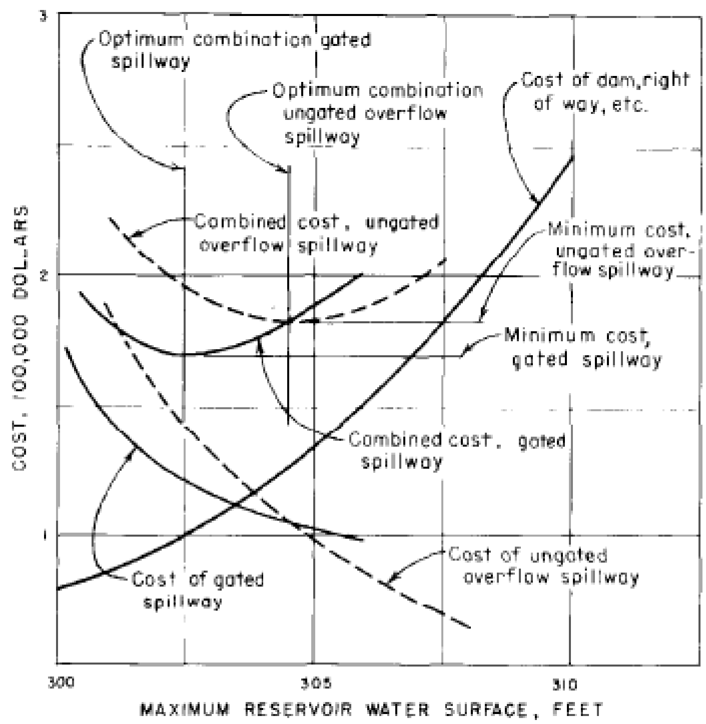


Figure 9-6.—Comparative costs of spillway-dam combinations. 288-D-2403.

(b) Combined Service and Auxiliary Spillways.-

Where site conditions are favorable, the possibility of gaining overall economy by using an auxiliary spillway in conjunction with a smaller service-type spillway should be considered. In such cases the service spillway should be designed to pass floods likely to occur frequently, and the auxiliary spillway control set to operate only after such small floods are exceeded. In certain instances the outlet works may be made large enough to serve also as a service spillway. Conditions favorable for the adoption of an auxiliary spillway are the existence of a saddle or depression along the rim of the reservoir that leads into a natural waterway or a gently sloping abutment where an excavated channel can be carried sufficiently beyond the dam to avoid the possibility of damage to the dam or other structures.

Because of its infrequent use, the entire auxiliary spillway need not be designed for the same degree of safety required for other structures. However, the control portion must be designed to forestall failure because its breaching would release large flows from the reservoir. For example, concrete lining may be omitted from an auxiliary spillway channel excavated in competent rock. Where the channel is excavated through less competent material, it might be lined but terminated above the river channel with a cantilevered lip rather than extended to a stilling basin at river level. The design of auxiliary spillways is often based on the premise that some damage to portions of the structure from passage of infrequent flows is permissible. Minor damage by scour to an unlined channel, by erosion and undermining at the downstream end of the channel, and by creation of an erosion pool downstream from the spillway may be acceptable.

An auxiliary spillway can be designed with a fixed crest control, or it can be stoplogged or gated to increase the capacity without additional surcharge head. Fuseplug dikes, which are designed to breach and wash out when overtopped, often are substituted for some or all of the gates. Their advantage over gates is that, if they are properly designed, breaching becomes automatic whenever overtopping occurs. Furthermore, they are cheaper to install and to maintain. Because the chance of their failure from overtopping depends on the occurrence of infrequent floods, their replacement cost is too problematical for evaluation. By dividing the dike into short sections of varying height, so they are not all simultaneously over-lopped, smaller floods can be passed with the failure of one or several of the sections; total failure will occur only as the probable maximum flood is approached. The breaching of one section at a time will minimize the flood wave and the possibility of a sudden failure of the dike (see [l]‘).

Figure 9-7 shows a general plan and sections of a service and an auxiliary spillway. Figure 9-8 is an aerial photograph showing this service spillway, which consists of a bathtub-shaped side channel crest, a culvert conduit under the dam, a diverging concrete-lined chute, and a hydraulic-jump stilling basin. Figure 9-9 is an aerial photograph showing the wide auxiliary spillway channel with fuse plug control (at the top of the figure) and the service spillway chute. Note the outlet works control house and the outlet works channel, which empties into the spillway stilling basin.

The aforementioned auxiliary spillway channel was excavated in a soft sandstone with only fair erosion-resistant qualities. To minimize erosion should discharge occur, the channel floor was made level so that velocities would be low. Erosion would start at the downstream end and progress slowly upstream. The control structure consists of a concrete- lined section; the cantilever lip and the downstream cutoff are provided to halt erosion upstream.

The division walls and the fuse plug sections of varying crest elevations ensure that a failure of the dike will be progressive. The two sections nearest the dam were made the highest so that they will be the last to be overtopped. This was done to keep the flows away from the dam and to make the channel flow distance longer for discharges less than the maximum for which the spillway was designed.

(c) Emergency Spillways.-As the name implies, emergency spillways are provided for additional safety should emergencies not contemplated by normal design assumptions arise. Such situations could be the result of an enforced shutdown of the outlet works, a mal-functioning of the spillway gates, or the necessity for bypassing the regular spillway because of damage or failure of some part of that structure. An emergency might arise where flood inflows are handled principally by surcharge storage and a recurring flood develops before a previous flood is evacuated by the small service spillway or the outlet works. Emergency spillways would act as auxiliary spillways if a flood greater than the selected inflow design flood occurred.

Under normal reservoir operation, emergency spillways are never required to function. Therefore, the control crest is placed at or above the designed maximum reservoir water surface. The freeboard requirement for the dam is based on a water surface determined by assuming an arbitrary discharge that might result from a possible emergency. Usually, an encroachment on the freeboard provided for the designed maximum water surface is allowed in considering the design of an emergency spillway.

Emergency spillways are provided primarily to avoid an overtopping of the main dam embankment because of an emergency condition. Therefore, to be effective the emergency spillway must offer resistance to erosion greater than does the dam itself.

Emergency spillways are often formed by lowering the crest of a dike section below that of the main embankment, by using saddles or depressions along the reservoir rim, or by excavating channels through ridges or abutments. The outlet channel of an emergency spillway should be far enough from the dam to preclude damage to the main embankment or appurtenances should the spillway operate.

Figure 6-84 shows an emergency spillway at Wasco Dam. This spillway was designed to prevent overtopping of the embankment should the combination outlet works-spillway fail to function properly.

B. SERVICE SPILLWAYS

9.8. Spillway Types.-(a) General.-Spillways are ordinarily classified according to their most prominent feature, either as it pertains to the control, to the discharge channel, or to some other feature.

Spillways are often referred to as “controlled” or “uncontrolled,” depending on whether they are gated or ungated. Commonly referred to types are the free overfall (straight drop), ogee (overflow), side channel, labyrinth, open channel (trough or chute), conduit, tunnel, drop inlet (shaft or morning glory), baffled apron drop, culvert, and siphon.

(b) Free Overfull (Straight Drop) Spillways.-A free overfall, or straight drop, spillway is one in which the flow drops freely from the crest. This type is suited to a thin arch or to a crest that has a nearly discharging, as is the case with a sharp-crested weir control, or they may be supported along a narrow section of the crest. Occasionally, the crest is extended in the form of an overhanging lip to direct small discharges away from the face of the overfall section. In free overfall spillways the underside of the nappe is ventilated sufficiently to prevent a pulsating, fluctuating jet.

Where no artificial protection is provided at the base of the overfall, scour will occur in most streambeds and will form a deep plunge pool. The volume and depth of the hole are related to the range of discharges, the height of the drop, and the depth of tail water. The erosion-resistant properties of the streambed material, including bedrock, have little influence on the size of the hole, they only effect the time necessary to scour the hole. Probable depths of scour are discussed in section 9.25. Where erosion cannot be tolerated, an artificial pool can

be created by constructing an auxiliary dam downstream from the main structure, or by excavating a basin, which is then provided with a concrete apron or bucket.

If tail water depths are sufficient, a hydraulic jump will form when a free overfall jet falls upon a flat apron. It has been demonstrated that the momentum equation for the hydraulic jump may be applied to the flow conditions at the base of the fall to determine the elements of the jump.

A free overfall spillway that will be effective over a wide range of tailwater depths can be designed for use with low earthfill dams [4]. An artist’s conception of such a structure is shown on figure 9-11.

It consists principally of a straight wall weir set at the upper end of a rectangular flume section, with its horizontal apron placed at or below streambed level. Floor blocks and an end sill are provided to help establish the jump and to reduce the downstream scour. This type of structure is not adaptable for high drops on unstable foundations because of the large impact forces that must be absorbed by the apron at the point of impingement of the jet.

Vibrations incident to the impact caused by high drops might crack or displace the structure, with danger of failure by piping or undermining. Ordinarily, the use of this structure for hydraulic drops from head pool to tail water of 20 feet or more should not be considered. The hydraulic design of the free overfall spillway is discussed in section 9.26.

(c) Ogee (Overflow) Spillways,-The ogee spillway has a control weir that is ogee-shaped (S-shaped) in profile. The upper curve of the ogee spillway ordinarily conforms closely to the profile of the lower nappe of a ventilated sheet falling from a sharp-crested weir. Flow over the crest adheres to the face of the profile by preventing access of air to the underside of the sheet. For discharges at designed head, the flow glides over the crest with no interference from the boundary surface and attains near-maximum discharge efficiency. The profile below the upper curve of the ogee is continued tangent along a slope to support the sheet on the face of the overflow. A reverse curve at the bottom of the slope turns the flow onto the apron of a stilling basin or into the spillway discharge channel.

The upper curve at the crest may be either broader or sharper than the nappe profile. A broader curve will support the sheet, and positive hydrostatic pressure will occur along the contact surface. The supported sheet thus creates a backwater effect and reduces the efficiency of discharge.

For a sharper curve, the sheet tends to pull away from the crest and to produce sub-atmospheric pressure along the contact surface. This negative pressure effect increases the effective head and, thereby, increases the discharge.

An ogee crest and apron may make up an entire spillway, such as the overflow portion of a concrete gravity dam, or the ogee crest may only be the control structure for another type of spillway. Because of its high discharge efficiency, the nappe-shaped profile is used for most spillway control crests. Crest shapes and discharge coefficients are discussed in sections 9.10, 9.11, and 9.12.

(d) Side Channel Spillways.-A side channel spillway is one whose control weir is placed alongside and approximately parallel to the upper portion of the spillway discharge channel. Flow over the crest falls into a narrow trough opposite the weir, turns approximately 90”, and then continues into the main discharge channel. The side channel design is concerned only with the hydraulic action in the upstream reach of the discharge channel and is more or less independent of the details selected for the other spillway components. Flows from the side channel can be directed into an open discharge channel or into a closed conduit or inclined tunnel.

Flow into the side channel might enter the trough on only one side in the case of a steep hillside location, or on both sides and over the end of the trough if it is located on a knoll or gently sloping abutment.

The bathtub-type side channel spillway shown on figures 9- 7, 9-8, and 9-9 illustrates the latter case. Figure 9-12 is an artist's conception of a side channel spillway where flow enters only one side of the trough.

Discharge characteristics of a side channel spillway are similar to those of an ordinary overflow spillway and are dependent on the selected profile of the weir crest. However, for maximum discharges the side channel flow may differ from that of the overflow spillway in that the flow in the trough may be restricted and may partly submerge the flow over the crest. In this case the flow characteristics are controlled by a constriction in the channel downstream from the trough. The constriction may be a point of critical flow in the channel, an orifice control, or a conduit or tunnel flowing full.

Although the side channel is neither hydraulically efficient nor inexpensive, it has advantages that make it desirable for certain spillway layouts.

Where a long overflow crest is needed to limit the surcharge head and the abutments are steep and precipitous, or where the control must be connected to a narrow discharge channel or tunnel, the side channel spillway is often the best choice.

The hydraulic design of side channel spillways is discussed in section 9.17.

(e) Labyrinth Spillways.-The concept behind the labyrinth spillway is to provide added crest length for a given total spillway width, so that less head is required to pass a given discharge. The additional spillway crest length is obtained by a series of trapezoidal or triangular walls within the total spillway width (see fig. 9-13). These walls are thin and cantilevered, vertical on the upstream face and steeply sloping (1:10 or 1:16) on the downstream face. They are supported with a concrete base slab or are tied into an existing good quality foundation.

The crest consists of a quarter-circle arc on the upstream edge and a slight chamfer on the downstream edge.

Labyrinth spillways have many advantages and applications. They are suitable for use anywhere an overflow structure is required depending upon the site conditions. A labyrinth design is particularly beneficial when the spillway width is fixed, upstream water surface elevations are restricted, and large discharges must be passed. The increased crest length produced by the labyrinth configuration allows passage of greater discharges under less head. Labyrinths are particularly suitable for use at a reservoir site, either as a service spillway or an auxiliary spillway. Where an inflow design flood has

been increased and the capacity of an existing spill abutment way must also be increased, a labyrinth spillway is an excellent alternative to traditional methods of adding another spillway. Labyrinths have also been used as control or diversion structures on canals.

Storage capacity can also be increased because the labyrinth crest can be set at a higher elevation than a straight crest while still passing the required discharge.

Labyrinth spillways are more economical than gated structures. Cost savings may be realized during initial construction and in future operation and maintenance costs. An example of the labyrinth spillway geometry and a typical application is shown on figure 9-13.

Flow patterns for the labyrinth spillway are very complicated. The primary parameters affecting flow patterns and, thus, spillway performance, are the length magnification, crest length per cycle width, the discharge and head over the spillway, the angle of the spillway side walls with respect to the flow, and the ratio of the spillway cycle width to the spillway height.

Ideally, discharge over the spillway should increase in direct proportion to the increase in crest length. However, this occurs only for small crest length to spillway width ratios and for small head to crest height ratios. Because labyrinth spillways are most advantageous when designed to operate under conditions that exceed these restrictions, analysis of spillway performance is complicated.

Basically, spillway performance is determined by the flow patterns in the upstream and downstream channels of each cycle. Therefore, the spillway geometry chosen must allow optimum flow distribution in these areas [5]. Hydraulic model studies for Hyrum Dam auxiliary labyrinth spillway and for Ute Dam labyrinth can be found in [6] and [7], respectively. (f) Chute (Open Channel or trough) Spillways.-

A spillway, whose discharge is conveyed from the reservoir to the downstream river level through an open channel, placed either along a dam abutment or through a saddle, might be called a chute, open channel, or trough spillway. These designations apply regardless of the control device used to regulate the flow. Thus, a spillway having a chute-type discharge channel, though controlled by an overflow crest, a gated orifice, a side channel crest, or some other control device, may still be called a chute spillway.

However, the name is most often applied when the spillway control is placed normal or nearly normal to the axis of an open channel, and where the streamlines of flow both above and below the control crest follow in the direction of the axis.

The chute spillway has been used more often with earth-fill dams than with any other type. Factors influencing the selection of chute spillways are the simplicity of their design and construction, their adaptability to almost any foundation condition, and the overall economy often obtained by the use of large amounts of spillway excavation in the dam embankment.

Chute spillways ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel. The simplest form of chute spillway has a straight center line and uniform width, such as that shown on figure 9-14. Often, either the axis of the entrance channel or that of the discharge channel must be curved to fit the alignment to the topography. If possible, the curvature is confined to the entrance channel because of the low approach velocities.

When the discharge channel must be curved, its floor is sometimes super elevated to guide the high velocity flow around the bend, thus avoiding a piling up of flow toward the outside of the chute.

Chute spillway profiles are usually influenced by the site topography and by subsurface foundation conditions. The control structure is generally placed in line with or upstream from the centerline of the dam. Usually the upper portion of the discharge channel is carried at minimum grade until it "daylights" along the downstream hillside to minimize excavation. The steep portion of the discharge channel then follows the slope of the abutment.

Flows upstream from the crest are generally at subcritical velocity, with critical velocity occurring when the water passes over the control. Flows in the chute are primarily maintained at supercritical stage, either at constant or accelerating rates, until the terminal structure is reached. For good hydraulic performance, abrupt vertical changes or sharp convex or concave vertical curves in the chute profile should be avoided. Similarly, the convergence or divergence in plan should be gradual to avoid cross waves, wave run up on the walls, excessive turbulence, or uneven distribution of flow at the terminal structure.

The hydraulic design of the chute spillway crest is discussed in part C, the determination of hydraulic properties for the discharge channel in part D, and stilling basin designs in part E of this chapter.

(g) Conduit and Tunnel Spillways.-Where a closed channel is used to convey the discharge around or under a dam, the spillway is often called a tunnel or conduit spillway, as appropriate. The closed channel may take the form of a vertical or inclined shaft, a horizontal tunnel through earth or rock, or a conduit constructed in open cut and backfilled with earth materials. Most forms of control structures, including overflow crests, drop inlet entrances, and side channel crests, can be used with conduit and tunnel spillways.

With the exception of those with drop inlet entrances, tunnel and conduit spillways are designed to flow partly full throughout their length. With the drop inlet, the tunnel or conduit size is selected so that it flows full for only a short section at the control and thereafter partly full for its remaining length. Ample aeration must be provided in a tunnel or conduit spillway to prevent a make-and-break siphonic action that would occur if some part of the tunnel or conduit sealed temporarily. This sealing could be the result of an exhaustion of air caused by surging of the water jet, or by wave action or backwater. To guarantee free flow in the tunnel, the ratio of the flow area to the total tunnel area is often limited to about 75 percent. Air vents should be provided at critical points along the tunnel or conduit to ensure an adequate air supply, which would preclude unsteady flow through the spillway.

Air slots may be appropriate in some instances to introduce air into the flow for prevention of cavitation where high velocity flow occurs. The Bureau of Reclamation has prepared model studies of air slots for the spillways at Blue Mesa and Glen canyon dams [8], Hoover Dam [9], and Yellowtail Dam [10] .Additional information on aeration of spillway flows is presented in [11].

Thnnel spillways may present advantages for dam sites in narrow canyons with steep abutments or at sites where there is danger to open channels from snow slides or rockslides. Conduit spillways may be appropriate at dam sites in wide valleys, where the abutments rise gradually and are far from the stream channel. Use of a conduit will permit the spillway to be located under the dam near the streambed.

(h) Drop Inlet (Shaft or Morning Glory} Spillways.- As the name implies, a drop inlet or shaft spillway is one in which the water enters over a horizontal lip, drops through a vertical or sloping shaft, and then flows to the downstream river channel through a horizontal or nearly horizontal conduit or tunnel. The structure is considered to comprise three elements: an overflow control weir, a vertical transition, and a closed discharge channel.

Where the inlet is funnel-shaped, this type of structure is often called a "morning glory," or "glory hole," spillway. Discharge characteristics of the drop inlet spillway may vary with a range of head. The control shifts according to the relative discharge capacities of the weir, the transition, and the conduit or tunnel.

For example, as the heads increase on a morning glory spillway, the control shifts from weir flow over the crest to tube flow in the transition and then to full pipe flow in the downstream portion.

Full pipe flow design for spillways, except those with extremely low drops, is not recommended. This is discussed in section 9.20(e).

A drop inlet spillway can be used advantageously at dam sites in narrow canyons where the abutments rise steeply or where a diversion tunnel or conduit~ is available for use as the downstream leg. Another advantage of this type of spillway is that near maximum capacity is attained at relatively low heads; this characteristic makes the spillway ideal for use where the maximum spillway outflow is to be limited.

This characteristic also may be considered disadvantageous, because there is little increase in capacity beyond the design head should a flood larger than the selected inflow design flood occur.

However, this would not be a disadvantage if this type of spillway were used as a service spillway in conjunction with an auxiliary or emergency spillway.

An artist’s conception of a drop inlet spillway is shown on figure 9-15. Figure 9-16 shows such a conduit under construction. The hydraulic design is discussed in section 9.26. Additional information on the design and performance of drop inlet spillways is given in [12, 13, and 14].

(i) Baffled Chute Spillways.-Baffled chutes, or aprons, are used in spillways where water is to be lowered from one level to another, and where a stilling basin is not desirable. The baffle piers partially obstruct the flow, dissipating energy as the water flows down the chute so that the flow velocities entering the downstream channel are relatively low, Advantages of baffled chutes include economy, low terminal velocities of the flows regardless of the height of the drop, spillway operation unaffected by downstream degradation, and effective stilling action without requirements for initial tail water depth.

The chute is normally constructed at a slope of 2: l or flatter, extending below the outlet channel floor. Chutes having slopes steeper than 2: l should be model tested [15, 16, 17] and their structural stability should be checked. The lower end of the chute should be constructed far enough below the channel floor to prevent damage from degradation or from scour.

Design capacities of baffled chutes have varied from less than 10 to over 80 ft?/s per foot of width.

At Conconully Dam, the scale model of the spillway baffled chute was designed to represent prototype discharge up to 78 ft^3/s per foot of width and to operate effectively at 150 ft^3/s per foot of width.

The completed spillway for Conconully Dam is shown on figure 9-17. The generalized design procedures discussed in this section were obtained from test results on several models of baffled chutes developed by the Bureau of Reclamation [15, 16, 17].

The typical hydraulic design procedure for a baffled chute drop spillway is given in the steps listed below, which relate to figures 9-18, 9-19 and 9-20.

(1) Determine the maximum expected discharge, Q.

(2) Determine unit design discharge q = Q/W, where W is the chute width. The chute width may depend on the upstream channel width, the downstream channel width, economy, topography, and frequency of discharge, as well as on maximum discharge.

Model studies have shown that a baffled chute spillway design for a large unit discharge can be based on a discharge about two-thirds of the maximum expected discharge.

However, the height of the chute sidewalls must be higher than the value determined in step (10). It is suggested that the height be increased by an amount equivalent to the critical depth of one-third the maximum expected discharge.

(3) The entrance velocity, V, should be as low as practical. Ideal conditions exist when the entrance velocity VI = f/iii -5 (curve D on fig. 9-20) for discharges up to 69 ft^3/s per foot of width. Velocities near or above critical, Vc = f/iii, (curve C on fig. 9-20), cause the flow to be thrown into the air after striking the first baffle pier. Proper flow conditions must be provided at

the entrance to the baffled apron because satisfactory performance of the entire structure may depend on proper entrance flow conditions.

(4) A vertical offset between the approach channel floor and the chute is used to establish a desirable uniform entrance velocity, VI. This offset varies with the installation.

A short-radius curve provides a crest on the sloping chute. The first row of baffle piers should be placed no more than 12 inches in elevation below the crest. Alternate rows should be staggered to provide a baffle pier below each space and a space below each baffle pier. An alternative entrance configuration, the Fujimoto entrance (fig. 9-19), has been used successfully on several structures where the design unit discharge exceeds 100 ft3S. If the Fujimoto entrance is used, hydraulic model studies should be performed to determine the optimum location for the first row of baffle piers.

(5) The baffle pier height, H, should be about 0.8Dc or 0.9Dc, where the critical depth for the rectangular chute Dc = $\sqrt[3]{q^2/g}$ (curve A on fig. 9-20). Baffle pier height is not a critical dimension, but it should not be less than recommended. For unit discharges greater than 60 ft^3/s, curve A on figure 9-20 may be extrapolated.

(6) Baffle pier widths and spaces should equal, preferably, about 1.5 H but not less than H.

Other baffle pier dimensions are not critical hydraulically. Suggested cross-sectional dimensions are given on figure 9-18.

(7) The spacing between the rows of baffle piers down the chute slope should be H divided by the slope, where the slope is given in decimal form. For example, a 2:1 slope (0.50 in decimal form) makes the row spacing equal to 2H parallel to the chute floor.

(8) The baffle piers are usually constructed with the upstream face normal to the chute floor surface; however, piers with vertical faces may be used. Vertical-faced piers tend to produce more splash and less bed scour, but the differences are minor.

(9) At least four rows of baffle piers are usually needed to establish full control of the flow (although spillways with fewer rows have occasionally operated successfully). As many additional rows as required beyond upstream. At least one row of baffles should be buried below the outlet channel grade to protect against scour. Additional rows of baffles should be buried as needed to protect against degradation.

(10) The chute training walls should be three times as high as the baffle piers measured normal to the floor. This wall height will contain the main flow and most of the splash. It is not necessary or practical to build the walls high enough to contain all the splash.

(11) Riprap should be placed at the downstream ends of the training walls to prevent erosion of the banks.

(j) Culvert Spillways.-A culvert spillway is a special adaptation of the conduit or tunnel spillway. It is distinguished from the drop inlet in that its inlet opening is placed either vertically or inclined upstream or downstream, and its profile grade is uniform or nearly uniform at any slope. The spillway inlet opening may be sharp-edged or rounded, and the approach to the conduit may have flared or tapered sidewalls with a level or sloping floor. If it is desired that the conduit flow partly full for all conditions of discharge, special precautions should be taken to prevent the conduit from flowing full; if full flow is desired, bell mouth or streamlined inlet shapes are provided.

Culvert spillways operating with the inlet un-submerged act similarly to an open channel spillway.

Those operating with the inlet submerged, but with the inlet orifice arranged so that full conduit flow is prevented, act similarly to an orifice-controlled drop inlet spillway or to an orifice-controlled chute spillway. Where priming action is induced and the conduit flows full, the operation will be similar to that of a siphon spillway.

When culvert spillways placed on steep slopes flow full, reduced or negative pressures prevail along the boundaries of the conduit. Large negative pressures may cause cavitation to the surfaces of the conduit or even its collapse. Where cracks or joints occur along the low-pressure regions, there is also the possibility of drawing in the soil surrounding the conduit. Culvert spillways, therefore, should not be used for high-head installations where large negative pressures can develop. Furthermore, the transition flow phenomenon, when the flow changes from partial flow to full stage, is accompanied by severe pulsations and vibrations that increase in magnitude with increased culvert fall. For these reasons, culvert spillways should not be used for hydraulic drops exceeding 25 feet.

For drops less than 25 feet, culvert spillways offer advantages over similar types because of their adaptability for either partial flow or full flow operation and because of their simplicity and economy of construction. They can be placed on a bench excavated along the abutment on a relatively steep side hill or they can be placed through the main section of the dam to discharge directly into the downstream river channel. As is the case with a drop inlet, a principal disadvantage of the culvert spillway is that it does not provide a safety factor against underestimation of the design flood because its capacity does not substantially increase with increase in head. This disadvantage would not apply if the culvert spillway were used as a service spillway in conjunction with an auxiliary or emergency spillway.

The hydraulic design and details for culvert spillways are discussed in section 9.27.

9.9. Controlled Crests.-(a) Gene&-The simplest form of control for a spillway is the free, or uncontrolled, overflow crest, which automatically releases water whenever the reservoir water surface rises above crest level. The advantages of the uncontrolled crest are the elimination of the need for constant attendance and regulation of the control devices by an operator and the freedom from maintenance and repairs of the devices.

A regulating gate or other form of movable crest control is required if a sufficiently long uncontrolled crest or a sufficiently large surcharge head cannot be obtained for the required spillway capacity. Such devices are also required if the spillway is to release storages below the normal reservoir water surface.

Selection of the type and size of the crest control device may be influenced by such conditions as the discharge characteristics of the device, the climate, frequency and nature of floods, winter storage requirements, flood control storage and outflow provisions, the need for handling ice and debris, and special operating requirements. Whether an operator will be in attendance during flood periods and the availability of electric power, operating mechanisms, operating bridges, etc., are also factors that could influence the type of control device selected.

Many types of crest control have been devised. The type selected for a specific installation should be based on a consideration of the factors noted above as well as economy, adaptability, reliability, and efficiency. Movable crests include such devices as flashboards, stoplogs, and drum gates. Regulating devices include vertical or inclined rectangular lift gates, wheel-mounted gates, roller-mounted gates, and radial gates. Radial gates and wheel mounted slide gates are most commonly used for large spillways.

For simplicity of design and operation, the simpler control devices are considered appropriate for spillways for small dams. These devices include flashboards, stoplogs, rectangular gates, and radial gates, which should be used whenever possible because they can be easily fabricated and obtained commercially.

(b) Flashboards and Stoplogs.-Flashboards and stoplogs can be used as a means of raising the reservoir storage level above a fixed spillway crest level when the spillway is not needed for releasing floods.

However, safety of dams considerations often preclude the use of these devices. Flashboards usually consist of individual wooden boards, or structural panels anchored to the crest; stoplogs are wooden boards or structural panels spanning horizontally between slots or grooves recessed into the sides of the supporting piers. To provide adequate spillway capacity, the flashboards or stoplogs must be removed before the floods occur, or they must be designed or arranged so that they can be removed while being overtopped. These devices should be used only where adequate removal is ensured.

Various arrangements of flashboards have been devised. Some must be placed and removed manually, some are designed to fail after being overtopped, and others are arranged to drop out of position either automatically or after being manually triggered when the reservoir exceeds a certain stage. Flashboards provide a simple economical type of movable crest device, and they have the advantage that an unobstructed crest is provided when the flashboards and their supports are removed.

However, flashboards have several disadvantages that greatly limit their adaptability.

Among these disadvantages are the following: (1) they present a hazard if not removed in time to pass floods, especially where the reservoir area is small and the stream drainage basin is subject to flash floods; (2) they require the attendance of an operator or crew and equipment for their removal, unless they are designed to fail automatically; (3) if they are designed to fail when the water reaches a predetermined stage, their operation is uncertain, and when they fail they release sudden and undesirably large outflows; (4) ordinarily, they cannot be placed back into position while flow is passing over the crest; (5) if the spillway functions frequently, the repeated replacement of flashboards may be costly; and (6) in some cases, they can be used only during low inflow periods.

Stoplogs are usually wooden beams or structural steel panel units stacked one upon the other to the desired height. They form a bulkhead that is supported in slots or in grooves recessed into the supporting piers at each end of the span. The spacing of the supporting piers depends on the material from which the stoplogs are constructed, the head of water acting against the stoplogs, and the handling facilities available for installing and removing them. Stoplogs that are removed individually as the need for increased discharge occurs are the simplest form of a crest gate.

Stoplogs can be an economical substitute for more elaborate gates where relatively close spacing of piers is not objectionable and where removal is required only infrequently. However, stoplogs that must be removed or installed in flowing water may require elaborate handling mechanisms that make them as costly as gates with attached hoists. A stoplogged spillway requires the attendance of an operating crew for removing and installing the stoplogs.

Furthermore, the arrangement may present a hazard to the safety of the dam if the reservoir is small and the stream is subject to flash floods, because the stoplogs must be removed in time to pass the flood.

(c) Rectangular Lift Gates.-Rectangular lift gates span horizontally in slots or grooves recessed into the supporting piers. Although these gates may be made of wood or concrete, they are often made of cast iron or fabricated structural steel. The supporting slots or grooves are generally placed vertically, and the gates are raised or lowered by an overhead hoist.

For sliding gates the vertical side members of the gate structure bear directly on support members anchored on the downstream side of the pier slot or groove; sealing is effected by the contact pressure.

The size of this type of installation is limited by the relatively powerful hoisting equipment required to operate the gate because of the sliding friction that must be overcome.

(d) Wheel- or Roller-Mounted Gates.-Where larger gates are needed, wheels or rollers can be mounted along each side of the rectangular lift gates to make a wheel- or roller-mounted gate. Water loads are carried through the wheels into vertical tracks anchored on the downstream side of the pier slot or groove. The use of wheels greatly reduces the amount of friction and thereby permits operation of the gate with a less powerful hoist.

(e) Radial Gates.-Radial gates are usually constructed of structural steel. They consist of a cylindrical segment supported by radial arms and trunnion pins. The center of curvature of the cylindrical segment is usually made coaxial with the common centerline of the trunnion pins so that the entire thrust of the water load passes directly through the trunnion pins; thus, only a small friction moment need be overcome in raising or lowering the gate. Hoisting loads then consist of only part of the gate weight, friction between side seals and pier walls, frictional moment at the pins, and static pressure head on bottom seal protection. The gate may be counterweighted to partially counterbalance its weight, which further reduces the required capacity of the hoist.

The small hoisting effort needed to operate radial gates makes hand operation practical for small installations that might require power if another type of gate is installed. The hoisting forces involved also make the radial gate more adaptable to operation by a relatively simple automatic control apparatus. Where a number of gates are used on a spillway, they may be automatically controlled to open incrementally at increasing reservoir levels. Or only one or two gates might be equipped with automatic controls, while the remaining gates would be operated by hand or power hoists. Small radial gates that may be hand or motor operated are available commercially.

C. STRUCTURAL DESIGN DETAILS

9.28. General.-The structural design of a spillway and the selection of specific structural details are generally performed after the spillway type has been selected, its components have been arranged, and the hydraulic design has been completed.

Usually, the foundation material of a spillway is not able to adequately resist the destructive action of high-velocity flows; therefore, a non-erodible lining must ordinarily be provided along the spillway waterway. Such a lining prevents erosion, reduces friction losses by providing smooth bounding surfaces for the channel (this also permits smaller hydraulic sections), and provides a relatively watertight conveyance channel for directing flow past the dam. Economy and durability most often favor concrete as the appropriate lining material for water conveyance structures.

A spillway may be constructed on almost any foundation capable of sustaining applied loads without undue deformation. Although it is not usually advisable, a spillway may be placed on the face of or through an earth-fill dam, provided design details are carefully selected to accommodate settlement and to prevent leakage from the structure.

The type of walls, linings, and associated structures of a spillway and its design details should depend on the nature of the foundation. For example, the design details for a spillway founded entirely on rock should differ from one constructed on softer material. Structural details should differ according to foundation bearing capacities, settlement or heave characteristics, and permeability and seepage features. Concrete walls, linings, and associated structures must be designed to withstand normal hydrostatic and earth loadings, movements caused by temperature changes, and unequal or large foundation movements. The design must also provide for handling leakage from the channel or under seepage from the foundation, which might cause saturation of the underlying materials and large uplift forces on the structure.

Subsequent sections discuss the structural designs and miscellaneous details of open channel spillways, including crest structures, walls, and channel linings. The structural designs of spillway conduits and tunnels are similar to those for outlet conduits and tunnels, which are discussed in chapter 10.

9.29. Crest Structures and Walls.-Spillway control structures and overflow crests against which reservoir heads act are essentially overflow dams, and spillway abutment structures or flanking dikes are similar to concrete non-overflow dams or earth-fill embankments. The design of earth-fill dams is discussed in chapter 6, and the design of overflow and non-overflow concrete dams is discussed in chapter 8.

The nature or type of confining side walls selected for open channel spillways should depend on the material upon which they are founded and on the loading to which they will be subjected. For spillway channels excavated in rock or firm material, where sloping the wall faces is permissible, a lining placed directly against the excavated slopes may provide sufficient stability for forming the channel sidewalls. Otherwise, self-supporting retaining walls of the gravity, cantilever, or counter forted type are required. A monolithic flume-type section whose walls are continuous with the floor and heels is often used.

The design of a gravity or reinforced concrete retaining wall for a spillway is similar to that for a gravity dam in that the stability against sliding and overturning and the magnitude and distribution of the foundation reaction resulting from the weight and applied loads must be determined. Methods of analyzing gravity structures for stability, including allowable sliding factors, and of determining foundation reactions are discussed in chapter 8. Suggested allowable bearing values are presented in appendix C.

Earth loadings can be assumed on the basis of equivalent fluid pressures. Figure C-l gives criteria for determining soil loadings on vertical and inclined walls using Coloumb’s theory of active earth pressure. Additional design criteria for concrete retaining walls are covered in “Design Criteria for Concrete Retaining Walls” [29]. Wall footings must be safeguarded against frost heave, and wall panels must be articulated to accommodate foundation yielding or unequal settlement. To avoid differential settlement in soft or yielding foundations, wall footing dimensions should be selected to minimize foundation load concentrations and to provide nearly uniform bearing reactions across the base areas.

Inlet channel and chute walls may be subjected to various combinations of loading. When flow is occurring through the spillway, hydrostatic loads on the channel side of the wall tend to offset the backfill loads. If, however, the fill has shrunk away from the walls, they may be subjected to full channel side water load before deflecting enough to gain support from the backfill. This condition is more likely to exist where the top of the wall inclines toward the backfill. On the other hand, when the reservoir is drawn down below the spillway level and there is no flow through the structure, the walls are subjected to full backfill loads without any support from water loads. The structural design of wall members must consider all these loading possibilities.

When the backfill is not expected to be tight against the wall to help support it against water pressures, an increase in the allowable stresses may be considered.

When permeable backfill is placed behind stilling basin walls or when the back of the wall is partly exposed to tail water, the water pressure resulting from tail water must be added to the backfill loading.

For higher spillway discharges, the water level inside the basin will be depressed by the profile of the jump, and an unbalanced hydrostatic load acting to overturn the walls will occur. Unbalanced water loads may also result from wave action. The design loading assumptions must recognize this condition of unbalanced pressures and the increased uplift forces when sliding and overturning analyses are considered.

9.30. Open Chunnel Linings-Floor slabs for articulated floor and wall systems are provided primarily to form a reasonably watertight protective surfacing over the channel to prevent erosion or damage to the foundation. During spillway flows, the floor may be subjected to hydrostatic forces from the weight of the water in the channel, to boundary drag forces caused by frictional resistance along the surface, to dynamic forces caused by flow impingement, to uplift forces caused by the reduction of pressure along the boundary surface, and to uplift pressure caused by leakage through joints or cracks. When there are no spillway flows, the floor is subjected to the action of the elements, including expansion and contraction caused by temperature variations, alternate freezing and thawing, and weathering and chemical deterioration; to the effects of settlement and buckling; and to uplift pressures brought about by under seepage or high ground-water conditions. Because evaluating the various forces that might occur and making the lining heavy enough to resist them is not always possible, the thickness of the lining is most often selected empirically, and under drains, anchors, cut offs, etc., are provided to stabilize the floor.

When a spillway channel is excavated in rock, the concrete slab is cast directly on the excavated surface. Anchor bars grouted into holes drilled into the rock may be provided to tie the slab to the foundation.

Slabs tied to the foundation should be provided with control or contraction joints to control cracking caused by expansion and contraction. Typical details for articulated slabs on rock are shown on figure 9-71. The anchorage increases the effective weight of the slab by the weight of foundation rock to which the anchors can be tied. Depth and spacing of anchors should depend on the nature of the bedrock and the design loading. Anchors should be large enough to support the weight of the foundation to which they are attached without exceeding the yield stress of the steel. A grid work of perforated under drains laid on a lean concrete pad in gravel-filled trenches should be provided to prevent a buildup of uplift under the slab. Rubber or polyvinyl chloride waterstops are generally provided at the joints.

Monolithic floor and wall systems for narrow structures serve the same purpose and are subjected to the same loads discussed for articulated structures.

However, design details and procedures vary because of the type of structure. The thickness of a monolithic slab is generally determined from backfill loads, water and uplift loads, and an elastic foundation analysis. Transverse joints should be located at approximately 25 to 50-foot spacing. Cutoffs and transverse drains are usually placed at these joints (fig. 9-71).

When a spillway channel is excavated through earth, the slab may be cast directly on the excavated surface, or an intervening pervious blanket may be required. The choice depends on the nature of the foundation as related to its permeability, susceptibility to frost heave, and heterogeneity as it may affect differential settlement. Because the slab is not bonded to the foundation, it will expand and contract and it must be restrained from creeping when it is constructed on a slope. This is best achieved by installing cutoffs (sec. 9.31(a)), which can be held relatively fixed with respect to the slab and to the foundation, or by tying the slab to walls, piles, or similar rigid members of the spillway structure.

Because a slab on an earth foundation is relatively free to move, the paving should be reinforced sufficiently to permit its sliding without cracking of the concrete or yielding of the reinforcement. To further assist in holding the slab to the foundation, bulb anchors are sometimes used, as shown on figure 9-71. These anchors, in effect, tie the slab to a cone of earth, the volume of which depends on the anchor depth and spacing and on the angle of internal friction of the soil.

A pervious gravel blanket is often provided between the slab and the foundation when the foundation is sufficiently impervious to prevent leakage from draining away, or where the foundation is subjected to capillarity, which will draw moisture to the underside of the lining. The blanket serves as a free-draining medium and helps insulate the foundation against frost penetration. Therefore, the thickness of the blanket selected should be based on the climate and on the susceptibility of the foundation to frost heaving. A grid work of perforated under drains laid in gravel and bedded on a lean concrete pad to prevent the foundation material from being leached into the pipe should be provided as a collection system for the seepage. The network of drainage pipe should empty into one or more trunk drains that carry the seepage flow to outlets through the channel floor or walls. In stratified foundations, ground water or seepage can cause uplift on layers below the floor lining, and drainage holes are sometimes augured into the underlying material and backfilled with gravels to relieve the under pressure.

When water tightness of the slab against exterior water heads is required, polyvinyl chloride or rubber waterstops should be installed to seal the joints. If water tightness is desired, such seals are provided in floor slabs upstream from the control structure to increase the percolation path under the structure.

They are commonly provided at transverse joints along concave curved portions of the downstream channel where the dynamic pressures on the floor cause a high head for introducing water into the joint. Seals may be desirable along longitudinal joints in a stilling basin on a permeable base. Differential heads resulting from the sloping water surface of the jump can cause a circulating flow under the slab if leakage is allowed to enter the joint at the downstream end of the basin and to flow out of the joint at the upstream end.’

Joints should generally be spaced from 25 to 50 feet apart in both the floor and walls. Joints should also be provided where angular changes of the floor surface occur and where they are required to avoid reentrant angles in the slab, which often cause cracking of the slab. The use of joint fillers in contraction joints should be minimized because deterioration of these fillers will result in an open joint that is difficult to maintain. If joints are provided at the proper spacings, contraction or expansion may not be severe, and filler material in the joint may not be necessary. Floor slabs can be constructed in alternate panels; the initial placement shrinkage of the concrete may then afford sufficient joint opening for subsequent expansion. A keyed joint in thin floors and walls that may be subjected to differential movement are unsatisfactory, because differential deflection across the joint places high stress on the keys or keyways and causes them to spall; an un-keyed joint with slip dowels is preferable.

Normally, the floor of a stilling basin will be subjected to uplift pressures resisting the tail water loads and water loads whose magnitudes depend on hydraulic-jump depths. For articulated slabs, the uplift pressure must be resisted by the weight of the slab and the water inside the basin and by anchor bars. Floors cast monolithically with walls experience uplift loads, inside water loads, and backfill loads and water loads transferred through the walls.

A transverse strip of the floor is usually analyzed with appropriate loadings and elastic foundation procedures. Flotation stability is computed assuming water to the elevation of the outlet channel and no water inside the basin.

9.3 1. Miscellaneous Details. - (a) Cutoffs. - One or more cutoffs are generally provided at the upstream end of a spillway for various purposes. They can be used to form a watertight curtain against seepage under the structure, or they can increase the path of percolation under the structure and thus reduce uplift forces. Cutoffs can also be used to intercept permeable strata in the foundation to minimize seepage and prevent a buildup of uplift pressure under the spillway or adjacent areas. When the cutoff trench for the dam extends to the spillway, it is generally joined to the upstream spillway cutoff to provide a continuous barrier across the abutment area. In jointed rock the cutoff acts as a grout cap for a grout curtain, which is often extended across the spillway foundation.

A cutoff is usually provided at the downstream end of a spillway structure as a safeguard against erosion and undermining of the end of the structure.

Cutoffs at intermediate points along the length of a spillway are sometimes provided as barriers against water flowing along the contact between the structure and the foundation and to lengthen the path of percolation under the structure.

Wherever possible, cutoffs in rock foundations are placed in vertical trenches. In earth foundations where the cutoffs must be formed in a trench with sloping sides, care must be taken to compact the trench backfill properly with impervious material to obtain a reasonably watertight barrier. (b) Backfill.-When a spillway is placed adjacent to a dam so that the impervious zone of the embankment abuts the spillway walls, the wall backfill is actually the impervious zone of the dam and should be compacted accordingly. Backfill elsewhere along the spillway walls should ordinarily be free-draining material to minimize hydrostatic pressures against the walls. Backfill other than that adjacent to the dam may be either compacted or uncompacted. The choice of backfill material and the compaction methods used in placing such material will affect the design loadings on the walls.

(c) Riprap.-When the spillway approach channel is excavated in material that will be eroded as a result of high approach velocities, a zone of riprap is often provided immediately upstream from the inlet lining to prevent scour of the channel floor and of the side slopes adjacent ot the spillway concrete.

This riprap, which is generally a continuation of that along the upstream face of the dam, should have similar size and gradation and similar bedding.

Riprap is normally used in the outlet channel adjacent to the downstream cutoff to prevent excessive erosion and undermining of the downstream end of the structure. To resist scour from high exit velocities, the riprap should be the largest possible and should be bedded on a graded material. The riprap should be graded to prevent the underlying material from washing out, which would cause the riprap to settle or to be displaced.

Outlet Works

A. GENERAL

10.1. Functions.-An outlet works regulates or releases water impounded by a dam. It can release incoming flows at a retarded rate, as does a detention dam; it can divert incoming flows into canals or pipelines, as does a diversion dam; or it can release stored waters at rates dictated by downstream needs, by evacuation considerations, or by a combination of multiple-purpose requirements.

Outlet works structures can be classified according to their purpose, their physical and structural arrangement, or their hydraulic operation. An outlet works that empties directly into a river could be designated a “river outlet”; one that discharges into a canal could be designated a “canal outlet”; and one that delivers water into a closed pipe system could be designated a “pressure pipe outlet.” An outlet works may be described according to whether it consists of an open-channel or closed-conduit waterway, or whether the closed waterway is a conduit in cut-and-cover or in a tunnel. An outlet works may also be classified according to its hydraulic operation: whether it is gated or ungated or, for a closed conduit, whether it flows under pressure for part or all of its length or only as a free flow waterway. Typical outlet works installations are shown on figures 10-l through 10-7.

Occasionally, the outlet works may be placed at a level high enough to deliver water to a canal, while a bypass is extended to the river to furnish necessary flows below the dam. Such bypass flows may be required to satisfy prior-right uses downstream or to maintain a live stream for abatement of stream pollution, preservation of aquatic life, or other purposes.

Dams constructed to provide reservoirs principally for recreation or for fish and wildlife conservation require a fairly constant reservoir level. For such dams an outlet works may be needed only to release the minimum flows necessary to maintain a live stream below the dam.

In certain cases, the outlet works of a dam may be used in lieu of a service spillway combined with an auxiliary or secondary spillway. In such a case, the usual outlet works installation might be modified to include a bypass overflow so that the structure can serve as both an outlet works and a spillway. Such structures are typified by Wasco Dam and Lion Lake dikes, figures 6-84 and 10-7(B), respectively. In these installations, the overflow weirs in the control shaft automatically bypass surplus inflows whenever the reservoir rises above normal storage level.

An outlet works may act as a flood control regulator to release waters temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows. Furthermore, the outlets may be used to empty the reservoir to permit inspection, to allow needed repairs, or to maintain the upstream face of the dam or other structures normally inundated. The outlets may also aid in lowering the reservoir storage when controlling or poisoning scrap fish or other objectionable aquatic life in the reservoir is desired.

10.2. Determination of Required Capacities. - Outlet works are designed to release water at specific rates. These rates are dictated by downstream needs, by flood control regulation, by storage considerations, by power generation needs (where the outlet works is used as the penstock for small power plants), and by legal requirements. Delivery of irrigation water is usually determined from project or farm needs and is related to the consumptive use and to the special water requirements of the irrigation system. Delivery for domestic use can be similarly established. Releases of flows to satisfy prior rights must generally be included with other needed releases. Minimum downstream flows for pollution abatement, fish preservation, and associated needs are often accommodated through other required releases.

A small bypass pipe is often used to provide these minimum releases. This pipe usually originates at the gate chamber or in the downstream control structure, depending on the type of outlet works.

Irrigation outlet capacities are determined from reservoir operation studies. They must be based on a consideration of a critical period of low runoff when reservoir storages are low and daily irrigation demands are at their peak. The most critical draft from the reservoir, considering such demands (commensurate with remaining reservoir storage) together with prior rights and other needed releases, generally determines the minimum irrigation outlet capacity. These requirements are stated in terms of discharge at either a given reservoir content or a given water surface elevation. Occasionally, outlet capacity requirements are established for several reservoir contents or alternative water surfaces. For example, outlet requirements may be set forth as 20 ft^3/s capacity at reservoir content 500 acre-feet, and 100 ft^3/s capacity at reservoir content 3,000 acre-feet.

Evacuation of water stored in an allocated flood control storage space of a reservoir can be accomplished through a gated spillway at the higher reservoir levels or through an outlet at the lower levels.

Flood control releases generally can be combined with the irrigation releases if the outlet empties into a river instead of into a canal. The capacity of a flood control outlet can be determined by the required time of evacuation of the given storage space, considering the inflow into the reservoir during the evacuation. Combined flood control and irrigation releases ordinarily must not exceed the safe channel capacity of the river downstream from the dam and must allow for all anticipated inflows immediately below the dam. These inflows may be natural runoffs, or the results of releases from storage developments along the river or from developments on tributaries emptying into the river.

If an outlet is to serve as a service spillway in releasing surplus inflows from the reservoir, the discharge required for this purpose may determine the outlet capacity. Similarly, the minimum outlet capacity can be determined by the discharge and the time required to empty the reservoir for inspection, maintenance, repair, or emergency drawdown. Here again, the inflow into the reservoir during the emptying period must be considered. The capacity at low reservoir level should be at least equal to the average inflow expected during the maintenance or repair period. It can, of course, be assumed that required repair will be delayed until service demands are light and that repairs will be made during low inflow and during seasons favorable to such construction.

An outlet works cut-and-cover conduit or tunnel is often used to divert the river flow during the construction period, precluding supplementary installations for that purpose. The outlet structure size dictated by this use, rather than the size dictated by ordinary outlet works requirements, may determine the final outlet works capacity. A diversion bypass pipe may be required to satisfy downstream requirements during placement of second-stage concrete and gates in the outlet works. 10.3. Outlet Works Position in Relation to Reservoir

Storage Levels.-The establishment of the intake level and the elevations of the outlet controls and the conveyance passageway, as they relate to the reservoir storage levels, are influenced by many factors. Primarily, to attain the required discharge capacity, the outlet must be placed sufficiently below the minimum reservoir operating level to provide the head required for outlet works flows.

Outlet works for small detention dams are generally constructed near riverbed level because permanent storage space, except for silt retention, is ordinarily not provided. (These outlet works may be ungated to retard the outflow while the reservoir temporarily stores the bulk of the flood runoff, or they may be gated to regulate the releases of the temporarily stored waters.) If the purpose of the dam is only to raise the reservoir and divert incoming flows at low heads, the main outlet works generally should be a head works or regulating structure at a high level. A sluiceway or small bypass outlet should also be provided to furnish water to the river downstream or to drain the water from behind the dam during off-season periods. Dams that impound water for irrigation, for domestic use, or for other conservation purposes, must have outlet works low enough to draw the reservoir down to the bottom of the allocated storage space; however, the outlet works may be placed above the riverbed, depending on the established minimum reservoir storage level.

It is common practice to make an allowance in a storage reservoir for inactive storage to accommodate sediment deposition, for fish and wildlife conservation, and for recreation. The positioning of the intake sill then becomes an important consideration; it must be high enough to prevent interference from the sediment deposits, but at the same time, low enough to permit either a partial or a complete drawdown below the top of the inactive storage.

As discussed in section 10.14, the size of an outlet conduit for a required discharge varies according to an inverse relationship with the available head for producing the discharge. This relationship may be expressed by the following equation:

$$H_T = K_1 h_v \text{ or } H_T = K_2 \frac{Q^2}{a^2} \qquad (1)$$

where:

HT = total available head for producing flow,

K_1 and K_2 = coefficients,

h, = velocity head,

Q = required outlet works discharge, and

a = required area of the conduit.

The above relationship for a particular design is shown on figure 10-8(A). This example shows that if the head available for the required outlet works discharge is increased from 1.6 to 4.6 feet, the corresponding conduit diameter can be decreased from 6 to 4.75 feet. This shows that the conduit size can be reduced significantly if the inactive storage level can be increased. The reduction in active storage capacity resulting from a 3-foot increase in the inactive storage level must be compensated for by the addition of an equivalent capacity to the top of the pool. The reservoir capacity curve on figure 10-8(B) shows that for equivalent storages (represented by de and gh), the 3 feet of head (represented by cd) added to obtain a reduced outlet works size would require a much smaller increase (represented by fg) in the height of the dam. Thus, economic studies can be used to determine the proper outlet size in relation to the minimum reservoir storage level.

Where an outlet is placed at riverbed level to accommodate the construction diversion plan (ch. 11) or to drain the reservoir, the operating sill may be placed at a higher level to provide a sediment and debris basin and other desired inactive storage space, or the intake may be designed to permit raising the sill as sediment accumulates. During construction, a temporary diversion opening may be formed in the base of the intake to handle diversion flows. Later, this opening may be plugged. For emptying the reservoir, a bypass around the intake may be installed at riverbed level. This bypass may either empty into the lower portion of the conduit or pass under it. Water can be delivered to a canal at a higher level by a pressure riser pipe connecting the conduit to the canal.

10.4. Conditions That Determine Out/et Works in a single structure. For example, the spillway and Layout.-The layout of an outlet works is influenced by many conditions relating to the hydraulic requirements, to the site adaptability, to the inter-relation of the outlet works and the construction procedures, and to the other appurtenances of the development. Thus, an outlet works leading to a high-level canal or into a closed pipeline might differ from one emptying into the river. Similarly, a scheme in which the outlet works is used for di version might vary from one where diversion is effected by other means. In certain instances, the proximity of the spillway may permit combining some of the outlet works and spillway components in a single structure. For example, the spillway and outlet works layout might be arranged so that discharges from both empty into a common stilling basin. An interesting arrangement in which a spill way and outlet works are combined in a single structure is shown on figure 10-9. In this installation, for Heart Butte Dam, the outlet works intake encircles the drop inlet tower of the spillway, and the outlet conduit extends along the top of the spillway conduit and empties into it downstream. Two other arrangements where the outlet works and spillway discharges empty into a common stilling basin, for Rifle Gap and Bottle Hollow dams, are shown on figure 10-9.

The topography and geology of a site may have a great influence on the layout selection. Some sites may be suited only for a cut-and-cover conduit type of outlet works; whereas, at other sites, either a cut and- cover conduit or a tunnel may be selected. Unfavorable foundation geology, such as deep overburdens or inferior foundation rock, precludes the selection of a tunnel scheme. On the other hand, sites in narrow canyons with steep abutments may make a tunnel outlet the only choice. Because of confined working space and excessive costs where hand-construction methods must be used, building a tunnel smaller than about 6 feet in diameter is not practicable. However, a cut-and-cover conduit can be built to almost any size if it is precast or cast-in-place with the inside bore formed by a prefabricated liner. Thus, the minimum size dictated by construction conditions, more than the size dictated by hydraulic requirements, influences the choice of either the cut-and-cover conduit or the tunnel scheme. The amount of load to be taken by a conduit will also affect this choice.

Some sites favorable for a tunnel outlet may have unfavorable portal conditions that make it difficult to fit the inlet and exit structures to the remainder of the outlet works. In this situation, a central tunnel with cut-and-cover conduits leading to and away from the tunneled portion of the outlet may be feasible. Such an arrangement is shown on figure 10-5 for McPhee Dam.

If water is to be taken from a reservoir for domestic use, or if temperature and heavy-metal control are required, special consideration must be given to the positioning of the intake. To ensure the proper quality of the water, it may be necessary to draw from different levels of the reservoir during different seasons or to restrict the draft to specific levels, depending on the reservoir stage. To prevent silt from being carried into the outlet system, intakes at low points or pockets in the reservoir must be avoided. Similarly, intakes must not be placed at points in the reservoir where stagnant water or algae can accumulate or where prevailing winds will drift debris or undesirable trash to the intake entrance.

10.5. Arrangement of Outlet Works.-The outlet works for a low dam, whether it is to divert water into a canal or release it to the river, often consists of an open-channel or cut-and-cover structure at the dam abutment. The structure may consist of a conventional open flume or rectangular channel with a gate similar to that used for ordinary spillway installations, or it may be regulated by a submerged gate placed to close off openings in a curtain or headwall. Where the outlet is to be placed through a low earth-fill embankment, a closed structure may be used. This structure may consist of single or multiple units of buried pipe or box culverts placed through or under the embankment.

Flow for such an installation could be controlled by gates placed at the inlet or at an intermediate point along the conduit, such as at the crest of the embankment, where a shaft would be provided for gate operation. Downstream from the control structure, the channel would continue to the canal or to the river where, depending on the exit velocities, a stilling device similar to one described in chapter 9 may be used. Figure 10-l shows typical installations of the arrangements described above.

For higher earth-fill dams, where an open-channel outlet structure would not prove feasible, the outlet might be carried through, under, or around the dam as a cut-and-cover conduit or through the abutment as a tunnel. Depending on the position of the control device, the conduit or tunnel may be free flowing, flowing under pressure for a portion of its length, or flowing under pressure for its entire length. Intakes may be arranged to draw water from the bottom of the reservoir, or the inlet sills may be placed at some higher reservoir level. Dissipating devices similar to those described in chapter 9 may be used at the downstream end of the conduit. The outlet works also may discharge into the spillway stilling basin. Depending on the method of control and the flow conditions in the structure, access to the operating gates may be by bridge to an upstream intake tower, by shaft from the crest level of the dam, by walkway within the conduit or tunnel with entrance from the downstream end, or by a separate conduit or tunnel access adit. Arrangements typical of those described above are shown on figures 10-2 through 10-5.

For a concrete dam, the outlet works installation should usually be carried through the dam as a formed conduit or a sluice, or as a pipe embedded in the concrete mass. Intakes and terminal devices may be attached to the upstream and downstream faces of the dam. Often, the outlet is formed through the spillway overflow section using a common stilling basin to dissipate both spillway and outlet works flows. Where an outlet works conduit is installed in the non-overflow section of the dam or where an outlet must empty into a canal, a separate dissipating device will, of course, be necessary.

Instead of one large conduit, several smaller conduits may be used in a concrete dam to provide a less expensive and more feasible arrangement for handling the outlet works releases. The multiple conduits may be placed at a single level or, for added flexibility, at several levels. Such an arrangement would reduce the cost of the control gates because of the lower heads on the upper-level gates. Typical outlet works installations for concrete dams are shown on figure 10-6.

A diversion tunnel used during the construction of a concrete dam can often be converted into a permanent outlet works by providing outlet sluices or conduits through the tunnel plug. Ordinarily, the diversion tunnel for a concrete dam will be in good quality rock and will therefore require little lining protection. Furthermore, the outlet portal of the tunnel will generally be located far enough downstream from the dam so that no dissipating structure will be needed or, at most, only a deflector will be required to direct the flow to the downstream river channel.

10.6. Location of Outlet Works Controls.- (a) General.-Where the outlet works is ungated, as is the case for many detention dams, flow in the conduit will be similar to that in a culvert spillway, as described in section 9.27. Where water must be stored and the release regulated at specific rates, control gates or valves must be installed at some point along the conduit.

Gates and valves for outlet works are categorized according to their function in the structure. Regulating gates and valves are used to control and regulate the outlet works flow and are designed to operate in any position from closed to fully open.

However, care should be taken in operating large gates at small openings because of potential cavitation problems. Guard gates are designed to effect closure only when the regulating gates fail or when unwatering is desired either to inspect the conduit below the guard gates or to inspect or repair the regulating gates. Generally, slots are provided at the conduit or tunnel entrance, and stop logs or bulkheads are stored nearby for use in the conduit or tunnel for inspection or during an emergency. For such installations, guard gates may or may not be provided, depending on whether or not the stop logs can be placed readily in an emergency during normal reservoir operating periods.

The control gate for an outlet works may be placed at the upstream end of the conduit, at an intermediate point along its length, or at the lower end of the structure. Where flow from a control gate is released directly into the open as free discharge, only that portion of the conduit upstream from the gate is under pressure. Where a control gate or valve is placed at the lower end of the structure, full internal pressure should be considered in the design of the conduit tunnel or pipe. However, when a control discharges into a free-flow conduit, the location of the control gate becomes important in the design of the outlet. The effects of locating the control at various positions in a conduit are discussed in the following subsections.

(b) Control at Upstream End of Conduit.-For an outlet works with an upstream control discharging into a free-flow conduit, partial full flow will occur throughout the length of the structure. Ordinarily, the operating head and the conduit slope will result in flow at the super-critical stage. The structural design of the conduit and the safety and practical aspects of the layout should then be concerned only with the effects of external loadings and of outside water pressures on the structure. Along the upstream portion of the conduit and extending until sufficient rock cover is available over a tunnel or until an adequate thickness of impervious embankment is obtained over a cut-and-cover conduit, practically full reservoir head will be exerted against the outside of the conduit barrel. The conduit walls must be designed to withstand such pressures, and the design details selected must preserve the water tightness of the conduit. For a cut-and cover conduit where settlement of the structure (caused by foundation consolidation with increasing embankment load) must be anticipated, special care must be taken in the design to prevent the cracking of the conduit barrel and to seal all formed joints. Cracks and open joints invite excessive leakage or piping of surrounding embankment material into the conduit.

With the controls placed at the upstream end of a conduit, fish screens, stop log slots, trash racks, guard gates, and regulating gates or valves may all be combined in a single intake structure. This arrangement simplifies outlet works operation by centralizing all control features at one point. Furthermore, the entire conduit may be readily un-watered for inspection or repair. The intake will consist of a tower rising from the base of the outlet conduit to an operating deck placed above maximum reservoir water level, with the tower located in the reservoir area near the upstream toe of the dam. Access to the structure operating deck will then be possible only by boat, unless an access bridge is provided from the reservoir shore or from the crest of the dam. The intakes at Crane Prairie and Crescent Lake dams (fig. 10-2) and McGee Creek and Palmetto Bend dams (fig. 10-4) illustrate typical tower arrangements. Figure 4-l is a photograph of the intake tower and access bridge at Crescent Lake Dam.

(c) Control at Intermediate Point along Conduit.- Where a control gate is placed at an intermediate point along a conduit and discharges freely into the downstream section or where the flow is conveyed in a separate downstream pipe, the internal pressure upstream from the control is approximately equal to full reservoir head. The structural design and safety aspects of the upstream portion will then be concerned with the effects of both the external loadings and the internal hydrostatic pressure acting on the conduit shell. The water tightness of the conduit in the extreme upstream section will be less important because the external and internal hydrostatic pressures will closely balance and leakage into or out of the conduit will be minimized. However, the external pressure around the conduit normally diminishes with increasing distance from the reservoir. At downstream portions of the pressure conduit, there may be excess internal pressure, which could cause leakage through joint or cracks into the material surrounding the conduit barrel. Such leaks may flow along the outside of the conduit to the section not under pressure where piping through joints could occur.

Where a pressure conduit is carried through an embankment, the development of piping, and the eventual failure of the dam, is a possibility. Where such a conduit is a tunnel, leakage through seams in the rock could saturate the hillside overburden above the tunnel and cause a sloughing or landslide on the abutment.

To minimize the possibilities of failures such as those described above, it is normal practice to limit the length of the pressure portion of a cut-and-cover conduit to that part of the outlet upstream from the crest of the dam or to approximately the upstream third of the dam. Where there is concern regarding the water tightness of a pressure conduit in the upstream portion of a dam, but there are compelling reasons why the control cannot be located near the upstream end of the conduit, that portion upstream from the control may be provided with a steel liner. This method was used at Sugar Loaf Dam (fig. 10-3).

For a tunnel installation, except for the possibilities of leakage discussed previously, the location of the control gate is not as critical as it is for a cut-and-cover outlet. However, the pressure portion of the tunnel ordinarily should not extend downstream beyond a point where the weight of the column of rock above the tunnel or the side resistance to a blowout is less than the internal pressure forces. The exception is where the tunnel lining is reinforced to withstand the internal pressure and a waterproof liner is provided to prevent a buildup of hydrostatic pressures outside the lining.

There may be instances where excessive settlement or movement of a conduit is expected and cracking and opening of joints cannot be avoided.

In this situation, to forestall serious leakage that would occur if a free flow or pressure conduit were used, a separate steel pipe can be installed inside the larger conduit to convey the flow. The control gate or valve is normally installed at the downstream end of such a pipe. Guard gates are normally provided in a chamber at the upstream end of the pipe to effect closure in the event of a leak or failure along any part of the pipe. See Silver Jack and Stateline dams on figure 10-3.

Where a control gate discharges into a free flow conduit, an access and operating shaft extending from the conduit to a level above the high water surface in the reservoir is required. For a cut-and cover conduit under an earth-fill dam, the location of the control gates should usually be selected so that the operating shaft is positioned immediately upstream from the crest of the dam. See McGee Creek and Twin Buttes dams on figure 10-4. The control gates or valves for a conduit or sluice through a concrete dam can be positioned at any point, either upstream to afford free flow in the sluice or at the downstream end to provide pressure pipe flow. Where the sluices are placed in the overflow section of the dam, upstream gates controlling the entrance or valves operated from an interior gallery in the dam are ordinarily used. Where the outlets are placed in the non-overflow section, either upstream gates or downstream valves are used (fig. 10-6).

Module III

Earth and rock fill Dams: subsurface explorations methods, cutoff trenches, sheet piling cutoffs, upstream blankets, horizontal drainage blankets and filters, toe drains and drainage trenches, pressure relief well. Seepage through embankments, Stability analysis of slopes of homogeneous and zoned embankment type under different reservoir conditions, Upstream and downstream slope protection measures.

Earth-fill Dams

A. INTRODUCTION

6.1. Origin and Development.-Earthfill dams have been used since the early days of civilization to store water for irrigation. This is attested both by history and by the remnants of ancient structures.

Some of the structures built in antiquity were very large. An earthfill dam completed in Ceylon in 504 B.C. [l]’ was 11 miles long, 70 feet high, and contained about 17,000,OOO yd3 of embankment. Today, as in the past, the earthfill dam continues to be the most common type of dam, principally because its construction involves using materials in their natural state with little processing.

Until modern times, all earthfill dams were designedby empirical methods, and engineering literature is filled with accounts of failures [2]. These failures brought on the realization that empirical methods must be replaced by rational engineering procedures for both the design and construction of earthfill dams. One of the first to suggest that the slopes for earthfill dams be selected on that basis was Bassell in 1907 [3]. However, little progress was made on the development of rational design procedures until the 1930’s. The rapid advancement of the science of soil mechanics since that time has resulted in the development of greatly improved procedures for the design of earthfill dams. These procedures include (1) thorough preconstruction investigations of foundation conditions and of construction materials, (2) application of engineering skill and technique to design, (3) carefully planned and controlled methods of construction, and (4) carefully planned and designed instrumentation and monitoring systems. Threaded throughout the plan, design, construct, operate, and maintain process is the philosophy that the design is not complete until the dam is accomplishing its purpose and has proved itself safe through several cycles of operation.

Earthfill dams have now (1987) been constructed to heights approaching 1,000 feet above their foundations, and hundreds of large rolled earthfill dams have been constructed in the past 40 years with a very good success record. Failures of small earthfill dams, however, occur more often. Though some of these failures are probably the result of improper design, many are caused by careless construction.

Proper construction methods include adequate foundation preparation and the proper placement of materials in the dam embankment-with the necessary degree of compaction and under established testing and control procedures.

The design of an earthfill dam must be realistic. It should reflect the actual foundation conditions at the site and the materials available for embankment construction. It should not be patterned after a successful design used at a site with different conditions or materials, or even at a site with similar conditions. It should be designed for its specific site geology.

6.2. Scope of Discussion.-This discussion is limited to design procedures for earthfill dams of the rolled-fill type of construction, as defined in section 6.3. This type of construction is now being used almost exclusively for the construction of earthfill dams. Semi-hydraulic or hydraulic fills are seldom, if ever, used. The information presented in this chapter is generally applicable to the design of any earthfill dam.

However, there are some empirical procedures presented that are strictly for the design of small dams, in straightforward geologic settings using trouble free embankment materials. A “small” dam is one whose maximum height above the lowest point in the original streambed does not exceed about 50 feet and whose volume is not so great that significant economic advantage would be obtained by using the more precise design methods usually reserved for large dams. A low dam cannot be considered small if its volume exceeds say, 1 million yd3. Figures 6-1 and 4-1 show typical small dams constructed by the Bureau (Bureau of Reclamation). Crane Prairie Dam, which was completed in 1940, has a height of 31 feet and contains 29,700 yd3 of fill. Crescent Lake Dam, which was completed in 1956, has a height of 22 feet and contains 16,800 yd3 of fill. The maximum sections of these dams are shown on figures 6-64 and 6-65, respectively.

Figures 6-2 and 6-3 show dams constructed by the Bureau that are at the upper limit of height for the use of the empirical procedures presented in this chapter. In fact, Fruitgrowers Dam (fig. 6-2) is slightly above the height limit. It has a maximum height of 55 feet and a volume of 135,500 yd3, but is included herein as a matter of interest. Irrigation at this site dates back to 1898. The dam shown on figure 6-2 was constructed in 1939, downstream from the original structure, which was breached in June 1937 to forestall failure. Fruitgrowers Dam was modified in 1986, to replace a damaged spillway and to increase flood bypass capacity and earthquake resistance. A maximum section of Fruitgrowers Dam is shown on figure 6-68. Many dams, small and large, are being modified to bring their capabilities up to modern-day requirements, especially in the area of flood capacity and earthquake resistance. Shadow Mountain Dam (fig. 6-3) is a 50- foot-high structure containing 168,000 ydi of embankment, which was completed in 1946. Its maximum section is shown in figure 6- 79.

The design procedures presented in this text are not sufficiently detailed to permit their sole use for the design of dams where complicated conditions such as exceedingly soft, exceedingly pervious, highly fractured, or collapsible soil foundations are involved. The design procedures are also inappropriate where the nature of the only soil available for construction of the embankment is unusual. In this category are dispersive soils, soils with high plasticity, with low maximum unit weight, and with very high natural water content that cannot be reduced by drainage. These conditions require that an engineer specializing in earthfill dam design direct the investigations, determine the laboratory testing program, interpret the laboratory test resuits, and supervise the preparation of the design and specifications.

6.3. Selection of Type of Earthfill Dam.-

(a) General.-The selection of the type of dam (earthfill, rockfill, concrete gravity, or a combination of these) is discussed in chapter 4. When the procedure leads to the selection of an earthfill dam, another decision must be made; that is, the type of
earthfill dam.

The scope of this text includes only the rolled fill type of earthfill dam. For this type, the major portion of the embankment is constructed in successive, mechanically compacted layers. The material from borrow pits and that suitable from required excavations for the dam and other structures is delivered to the embankment, usually by trucks or scrapers. It is then spread by motor graders or bulldozers and sprinkled, if necessary, to form lifts of limited thickness having the proper moisture content. These lifts are then thoroughly compacted and bonded with the preceding layer by means of power rollers of the proper design and weight.

Rolled-fill dams consist of three types: diaphragm, homogeneous, and zoned.

(b) Diaphragm Type.-For this type of section, most of the embankment is constructed of pervious (permeable) material (sand, gravel, or rock), and a thin diaphragm of impermeable material is provided to form the water barrier. The position of this impervious diaphragm may vary from a blanket on the upstream face to a central vertical core. The diaphragm may consist of earth, port land cement concrete, bituminous concrete or other material.

An earth blanket or core is considered a diaphragm if its horizontal thickness at any elevation is less than 10 feet or its thickness at any elevation is less than the height of the embankment above that elevation.

If the impervious earth zone equals or exceeds these thicknesses, the design is considered a zoned embankment type. Design and construction of diaphragm-type dams must be approached with care.

Although successful dams have been constructed with internal (or buried) diaphragms, this type of construction is not recommended for structures within the scope of this text. All internal diaphragms, including those constructed of earth or rigid materials such as concrete, have a potential for cracking caused by differential movements induced by embankment consolidation, fluctuating reservoir levels, and non-uniform foundation settlement.

The construction of an internal earth diaphragm with the necessary filters requires a higher degree of precision and closer control than that normally used for small dams. Internal diaphragms made of rigid material such as concrete also have the disadvantage of not being readily available for inspection or emergency repair if they are ruptured by settlement of the dam or its foundation.

An earth blanket on the upstream slope of an otherwise pervious dam is not recommended because of the expense and the difficulty of constructing suitable filters. Furthermore, because the earth blanket must be protected from erosion by wave action, it must be buried and therefore, is not readily available for inspection or repair. If the supply of impermeable soil is so limited that a zoned embankment dam cannot be constructed, a diaphragm of manufactured material placed on the upstream slope of an otherwise pervious embankment is recommended for small dams. The design of suitable impervious pavings is discussed in chapter 7.

If most of the material in a diaphragm-type dam is rock, the dam is classified as a rockfill dam. The design of rockfill dams is discussed in chapter 7.

(c) Homogeneous 1Ype.-A purely homogeneous dam is composed of only one kind of material (exclusive of the slope protection). The material used in such a dam must be sufficiently impervious to provide an adequate water barrier, and the slopes h a horizontal drainage blanket or pervious zones. Reliance should not be placed solely upon pipe drains because the pipes can clog as the result of improper filters, root growth, or deterioration.

Because drainage modifications to a homogeneous section provide a greatly improved design, the fully homogeneous section should seldom be used must be relatively flat for stability. To avoid sloughing, the upstream slope must be relatively flat if rapid drawdown of the reservoir after long-term storage is anticipated. The downstream slope must also be relatively flat to provide a slope stable enough to resist sloughing when saturated to a high level. For a completely homogeneous section, it is inevitable that seepage will emerge on the downstream slope regardless of its flatness and the impermeability of the soil if the reservoir level is maintained for long enough. The downstream slope eventually will be affected by seepage to a height of roughly one-third the depth of the reservoir pool [4], as shown on figure 6-4.

Although formerly very common in the design of small dams, the completely homogeneous section has been replaced by a modified homogeneous section in which small amounts of carefully placed pervious materials control the action of seepage so as to permit much steeper slopes. The effect of drainage at the downstream toe of the embankment is shown on figures 6-5(A) and 6-5(B).

Large rock toes may be provided for drainage (fig. 6-5(A)), or, if suitably graded materials are available, a horizontal drainage blanket (fig. 6-5(B)) may be used. The drainage and filter layers must be designed to meet filter requirements with surrounding fill or foundation materials (see sec. 6.10(i)). Recently, to avoid construction defects such as loose lifts, poor bond between lifts, inadvertent pervious layers, desiccation, and dispersive soils, inclined filter drains in combination with a horizontal drainage blanket have become almost standard. Figure 6-5(C) illustrates the control of seepage with an inclined chimney drain and horizontal drainage blanket.

Another method of providing drainage has been the installation of pipe drains. These are recommended for small dams only when used in Conjunction with a horizontal drainage blanket or pervious zones. Reliance should not be placed solely upon pipe drains because the pipes can clog as the result of improper filters, root growth, or deterioration.

Because drainage modifications to a homogeneous section provide a greatly improved design, the fully homogeneous section should seldom be used. Filtering and drainage should normally be provided.

A homogeneous (or modified homogeneous) dam is recommended in localities where readily available soils show little variation in permeability, and soils of contrasting permeabilities are available only in minor amounts or at considerably greater cost.

A homogeneous section should never be used if the available materials are dispersive, erodible such as silts and line sands, or subject to moderate to severe desiccation. Soils should always be tested for these characteristics. Where these characteristics exist, the advice of an experienced earthfill dam designer is recommended.

In any case, filter criteria given in section 6.10(i) must be met between the impervious zone and the downstream shell and between the shell and the foundation. For most effective control of through seepage and drawdown seepage, the permeability should progressively increase from the center of the dam out toward each slope.

(d) Zoned Embankment Type.-The most common type of a rolled earthfill dam section is that in which a central impervious core is flanked by zones of materials considerably more pervious, called shells. These pervious zones or shells enclose, support, and protect the impervious core; the upstream pervious zone affords stability against rapid drawdown; and the downstream pervious zone acts as a drain to control seepage and lower the phreatic surface. In many cases, a filter between the impervious zone and downstream shell and a drainage layer beneath the downstream shell are necessary. These filter-drainage layers must meet filter criteria with adjacent fill and foundation materials. They are sometimes multilayered for capacity requirements.

The pervious zones may consist of sand, gravel, cobbles, rock, or mixtures of these materials. For purposes of this text, the dam is considered to be a zoned embankment if the horizontal width of the impervious zone at any elevation equals or exceeds the height of embankment above that elevation in the dam and is at least 10 feet. The maximum width of the impervious zone will be controlled by stability and seepage criteria and by the availability of material. A dam with an impervious core of moderate width composed of strong material and with pervious outer shells may have relatively steep outer slopes, limited only by the strength of the foundation, the stability of the embankment itself, and maintenance considerations. Conditions that tend to increase stability may be decisive in the choice of a section even if a longer haul is necessary to obtain required embankment materials. If a variety of soils are readily available, the type of earthfill dam chosen should always be the zoned embankment because its inherent advantages will lead to more economical construction.

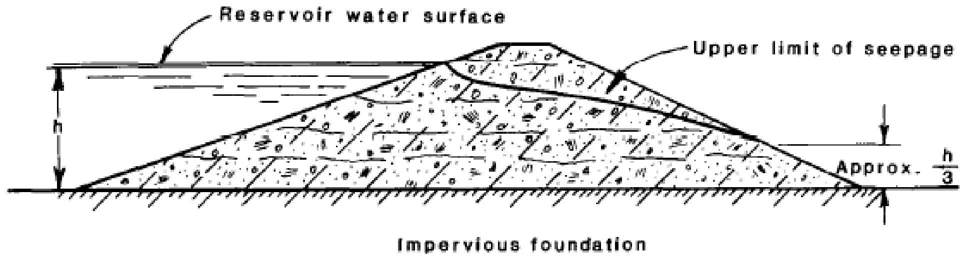


Figure 6-4.—Seepage through a completely homogeneous dam. 288-D-2479.

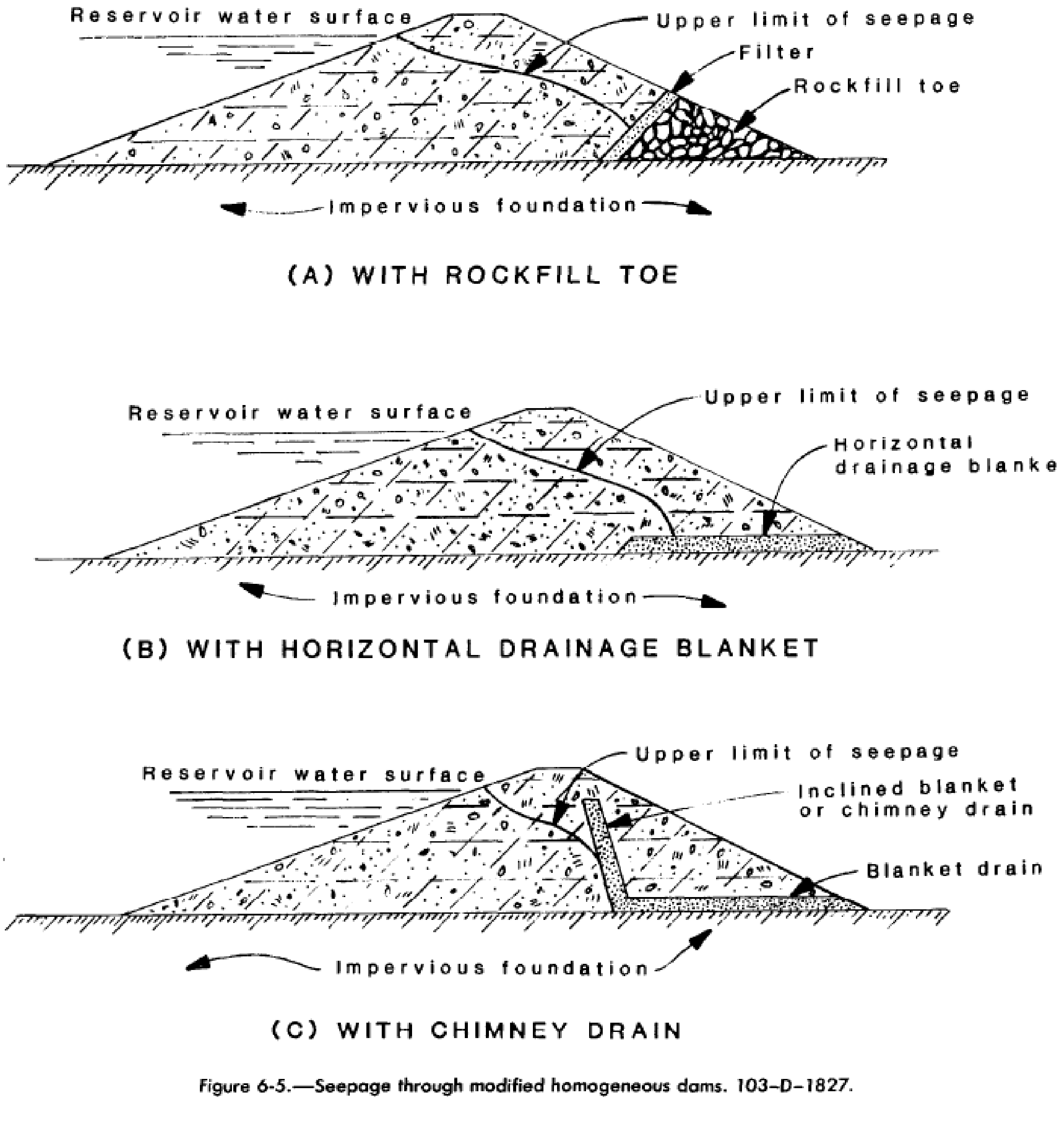


Figure 6-5.—Seepage through modified homogeneous dams. 103-D-1827.

B. DESIGN PRINCIPLES

6.4. Design Data. -The data required for the design of an earthfill dam are discussed in the various chapters of this manual, and the investigation of foundations and sources of construction materials are described in chapter 5. The required detail and the accuracy of the data are governed by the nature of the project and the immediate purpose of the design; that is, whether the design is for a cost estimate to determine project feasibility, whether the design is for construction, or whether some other purpose is to be served. The extent of investigations of foundations and sources of construction material are also governed by the complexity of the situation.

6.5. Design Criteria.-The basic principle of design is to produce a satisfactory, functional structure at a minimum total cost. Consideration must be given to maintenance requirements so that savings achieved in the initial cost of construction do not result in excessive maintenance costs. Maintenance costs vary with the provisions of upstream and downstream slope protection, drainage features, and the type of appurtenant structures and mechanical equipment. To achieve minimum cost, the dam must be designed for maximum use of the most economical materials available, including materials excavated for its foundations and for appurtenant structures.

An earthfill dam must be safe and stable during all phases of the construction and the operation of the reservoir. To accomplish this, the following criteria must be met:

(a) The embankment, foundation, abutments, and reservoir rim must be stable and must not develop unacceptable deformations under all loading conditions brought about by construction of the embankment, reservoir operation, and earthquake.

(b) Seepage flow through the embankment, foundation, abutments, and reservoir rim must be controlled to prevent excessive uplift pressures; piping; instability; sloughing; removal of material by solutioning; or erosion of material into cracks, joints, or cavities. The amount of water lost through seepage must be controlled so that it does not interfere with planned project functions.

(c) The reservoir rim must be stable under all operating conditions to prevent the triggering of a landslide into the reservoir that could cause a large wave to overtop the dam.

(d) The embankment must be safe against overtopping or encroachment of freeboard during occurrence of the IDF (inflow design flood) by the provision of sufficient spillway and outlet works capacity.

(e) Freeboard must be sufficient to prevent overtopping by waves.

(f) Camber should be sufficient to allow for settlement of the foundation and embankment, but not included as part of the freeboard.

(g) The upstream slope must be protected against wave erosion, and the crest and downstream slope must be protected against wind and rain erosion.

An earthfill dam designed to meet the above criteria will prove permanently safe, provided proper construction methods and control are achieved. The design procedure to meet the requirements of criterion (d) above is discussed in chapters 9 and 10.

Methods for satisfying other criteria for earthfill dams, subject to the limitations in scope described in section 6.2, will be discussed in this chapter. The applicability of the procedures to a specific case depends upon the purpose of the design, the size and importance of the structure, and the complexity of the problems.

C. FOUNDATION DESIGN

6.6. General.-The term “foundation” as used herein includes both the valley floor and the abutments.

The essential requirements of a foundation for an earthfill dam are that it provides stable support for the embankment under all conditions of saturation and loading, and that it provide sufficient resistance to seepage to prevent excessive loss of water.

Although the foundation is not actually designed, certain provisions for treatment are made in designs to ensure that the essential requirements are met.

No two foundations are exactly alike; each foundation presents its own separate and distinct problems requiring corresponding special treatment and preparation. Various methods of stabilization of weak foundations, reduction of seepage in pervious foundations, and types and locations of devices for the interception of under seepage must depend upon and be adapted to local conditions. The importance of adequate foundation treatment is emphasized by the fact that approximately 40 percent of all earthfill dam accidents and 12 percent of all failures are attributed to foundation failures.

Theoretical solutions based on principles of soil mechanics can be made for problems involving pervious or weak foundations. Most of these solutions are relatively complex and they may be relied upon only to the degree that the actual permeabilities in various directions or the strength of the foundation can be determined by expensive, detailed field and laboratory testing. Ordinarily, extensive exploration of this nature and complex theoretical designs are not required for small dams. For these structures, it is usually more economical to design foundations empirically, deliberately striving for substantial safety factors. The savings in construction costs that can be achieved by more precise design ordinarily do not warrant the cost of the additional exploration, testing, and engineering involved.

There are foundations, however, where conditions are so unusual that empirical methods cannot be relied upon to produce a design with an adequate safety factor. Such conditions require the services of an engineer specializing in the field of earthfill dam design and aie beyond the scope of this text.

Because different treatments are appropriate for different conditions, foundations are grouped into three main classes according to their predominant characteristics:

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

Foundations, which originate from various sources, such as river alluvium, glacial outwash, talus, and other processes of erosion, disintegration, and deposition, are characterized by infinite variations in the combinations, structural arrangement, and physical characteristics of their constituent materials. The deposits may be roughly stratified, containing layers of clay, silt, fine sand and gravel, or they may consist of lenticular masses of the same material without any regularity of occurrence and of varying extent and thickness. Nevertheless, the character of a foundation, as revealed by exploration, can usually be safely generalized for the design of small dams to fit into one of the classes given above, and once the class is determined the nature of the problem requiring treatment will be evident.

Ordinarily, coarse-grained, pervious foundations present no difficulties in the matter of settlement or stability for a small dam; conversely, fine grained, weak foundations subject to settlement or displacement usually present no seepage problems.

The special treatments required for the different types of foundations listed above are discussed in this chapter. If the foundation material is impervious and comparable with the compacted embankment material in structural charactertistics, little foundation treatment is required. The minimum treatment for any foundation is stripping the foundation area to remove sod, topsoil with high content of organic matter, and other unsuitable material that can be disposed of by open excavation. In many cases where the overburden is comparatively shallow, the entire foundation is stripped to bedrock. In all soil foundations in which a cutoff trench or partial cutoff trench (see sec. 6.10) is not used, a key trench should be provided. The top several feet of the soil foundation invariably lack the density of the underlying soil because of frost action, surface runoff, wind, or other cause. This layer should be penetrated by the key trench to allow inspection and to ensure cutoff by the impervious zone of the embankment through this questionable zone. A bottom width of 20 feet for the key trench is usually sufficient. The foundation at any particular site usually consists of a combination of the three main types of foundations listed above. For example, the stream portion often is a sand-gravel foundation, while the abutments are rock that is exposed on the steep slopes and mantled by deep deposits of clay or silt on the gentle slopes. Therefore, the design of any dam may involve a variety of foundation design problems.

4.7. Rock Foundations.-Rock foundations are generally considered to be the more competent type of foundation and usually do not present any problem for small dams. Even foundations of weaker rock are generally preferred over soil foundations.

The selection of a rock foundation is undoubtedly justified where the rock mass is generally homogeneous and competent throughout zones of the foundation that will be affected by the dam and reservoir. However, dam sites with good rock foundations are becoming increasingly rare. Designers are being forced to use foundations that are far from ideal because of the growth and shifting of population centers that cause increased emphasis on water conservation for domestic, agricultural, and industrial use in new locations. Rock foundations should be carefully investigated to ensure that they are adequately competent. If there is any doubt, an experienced earth dam designer should be consulted.

Foundation rock surfaces against which fill is to be placed must be properly treated to ensure that fractures, fault zones, steep faces, rough areas, weathered zones, etc., do not lead to seepage and piping in the interface zone between foundation and fill. Treatment of deficient foundation zones is especially critical for the areas beneath the impervious core and the filter and drainage zones immediately downstream of the impervious zone.

More explicit foundation surface treatment requirements are presented in chapter 3 of USBR Design Standard No. 13.

6.8. Methods of Treating Rock Foundations.-

Rock foundations should be carefully investigated to determine their permeability. If erosive leakage, excessive uplift pressure, or high water losses can occur through joints, fissures, crevices, permeable strata, or along fault planes, consideration should be given to grouting the foundation. Whether or not a foundation should be grouted should be determined by examining the site geology and by analyzing the water losses through foundation exploration holes. A great deal of experience is required to make this decision because every foundation is unique. Moreover, there may be more effective or economical methods of controlling seepage or leakage than grouting. The advice of an experienced designer should be sought when questionable conditions exist.

Ordinarily, the design and estimate for a storage dam should provide for foundation grouting. On the other hand, grouting of rock foundations is not generally required for small detention dams or for extremely low diversion and storage dams.

Foundation grouting is a process of injecting under pressure a fluid sealing material into the underlying formations through specially drilled holes to seal off or fill joints, fractures, fissures, bedding planes, cavities, or other openings. Unless the geologic conditions dictate otherwise, the foundation should be grouted to a depth below the surface of the rock equal to the reservoir head above the surface of the rock.

The grouting of a dam foundation is usually performed along a single line of grout holes spaced 10 to 20 feet on center. This creates some tightening deep in the foundation and some reduction in permeability. However, multiple lines of grout holes are necessary when severely fractured or highly permeable rock is encountered. Only multiple-line curtains improve the degree of reliability, but even then results are speculative because it is impossible to thoroughly grout all fractures or pores in the foundation. A grout curtain should not be relied on as the single provision to reduce seepage and related uplift pressures so that downstream seepage control features are reduced or eliminated. The grout curtain used on the abutment of Granby Dam in Colorado is shown on figure 6-6.

In cases where large zones of fractured rock lie at the foundation contact or where the zone of broken rock within a fault has great width, it may be possible to grout the zone by grouting to a shallow depth, usually 10 to 30 feet, by using a grid pattern.

This type of grouting is referred to as “blanket grouting.” It reduces leakage in the fractured zone and provides a more firm foundation for the dam.

In most cases, the foundation directly beneath the impervious zone requires some blanket grouting.

Foundation grouting is generally performed with a mixture of cement and water, starting with a ratio of 51. If considerable “take” in a hole is experienced, the grout mixture is progressively thickened.

Grout mixes usually vary between 1O: l and O.8: l. If the grout take is excessive, sand is added to give the gout additional bulk. In some cases, bentonite is combined with the sand in small quantities, about 2 percent by weight of the cement, to obtain a more pumpable grout mix and some expansion of the grout.

Where the grout hole continues to take a large quantity of grout, it may be advantageous to require intermittent pumping, waiting up to 24 hours between pumping periods to allow grout in the foundation to set.

Grouting is usually performed by one of the following methods: (1) staging-down, or (2) staging up. Grouting by the staging-down method consists of drilling the grout hole to a predetermined depth, washing the hole, pressure testing it with water, and then grouting. After grouting but before the grout in the hole has set, the grout is washed out of the hole and drilling for the second stage is begun. In the second and succeeding stages, the same sequence of operations is used, except that a packer is sealed near the bottom of the previously grouted stage. In this manner, subsequent stages are grouted until the entire length of the hole has been grouted.

This method is useful when drill-hole caving occurs, when the upper layers of the foundation are extensively cracked, or when the hole suddenly loses drill water.

When grouting by the staging-up method, the entire length of the hole is drilled, the hole washed, and a packer attached to the end of the grout supply pipe, which is then lowered and seated at a predetermined distance above the bottom of the hole.

Then grouting is performed at the required pressure. The grout pipe and packer are withdrawn to the next stage and the grouting is repeated. This upward staging continues until the entire hole is grouted.

Grout holes are usually drilled with the commercial standard EX (approximately 1%inch diameter) drill size, and a grout nipple is used to introduce the grout into the foundation. The grout nipple is usually a 2-inch-diameter pipe from 18 inches to 5 feet long (depending on rock conditions) that is anchored into the rock by cement grout, oakum, or other suitable calking material to facilitate drilling and grouting. The different drilling methods include air and water percussion and air and water rotary (plug or core bit). The primary concern when choosing a grout-hole drilling method is plugging fractures with cuttings. The drilling method should be chosen on the basis of the geologic conditions determined from data obtained during the design explorations.

Packers are devices that seal off drill holes at any elevation to permit grouting of a selected stage below the packer. The four types of packers most commonly used are shown on figure 6-7 and 6 -8.

The leather-cup packer (fig. 6-7(A)) seals when the grout forces the cups outward against the drill-hole wall; it is most commonly used in hard rock. The mechanical packer (fig. 6-7(B)) requires a double pipe arrangement; it is seated against the drill-hole wall by compressing the annular rubber sleeve at the bottom of the packer pipe by tightening the nut at the top of the pipe; this type of packer is more suitable than the leather-cup packer in slightly oversized holes. The pneumatic packer (fig. 6-8(C)) is expanded by compressed air or inert gas; it is used in poor rock where the drill holes may be considerably oversized. The cone-type packer (fig. 6- B (D)) is seated when grout forces the annular rubber sleeve upward on the cone; it is used in relatively hard rock. Photographs of the four types of packers are shown on figure 6-9.

A great variety of grouting equipment is available.

In general, the equipment consists of a grout mixer, grout agitator, grout pump, and a pipe and/ or hose system for circulating the grout. The circulating line and manifold system allows grouting pressures to be controlled at the collar of the hole.

Figure 6-10 illustrates the circulating-type grout system and the equipment generally used for grouting. Grout is usually pumped with a duplex piston type pump or a helical-screw rotor-type pump; a standby grout pump should always be required for the grout plant. Piston-type pumps require devices to smooth the pressure pulsations that occur at various phases of the stroke. Figure 6-11 shows the grout plant used at Ruedi Dam, Colorado.

Grouting pressures are influenced by the following factors:

- Type of rock
- Degree to which rock is fractured

- Jointing system within the rock
- Stratification of rock
- Depth of zone being grouted
- Location of hole being grouted
- Weight of overlying material at time of grouting

The maximum grouting pressure should be such pressures may weaken the rock strata by fracture, or may rupture a portion of the grout curtain already constructed, and result in increased permeability.

Maximum pressures are difficult to determine because each foundation has a unique rock joint pattern and stratification, which is usually found by trial at the actual time of foundation grouting or by performing grouting tests before foundation treatment.

Unless other criteria are established, l-lb/in^2 per foot of depth measured from the surface of the foundation to the center of the zone being grouted may be used as the initial grouting pressure. Variations may be determined by observing the grout take.

Current Bureau of Reclamation requirements for termination of grouting are presented in section G.60.

Grout should usually be introduced into the foundation through grout nipples set directly in the rock. Bedrock found to be badly jointed or broken below its surface may require a concrete group cap to facilitate grouting. However, use of a permanent grout cap can usually be avoided by leaving the foundation high and grouting through temporary grouted or concreted nipples or concrete caps. The use of grout caps under earth dams should be avoided because of the difficulty in sealing between them and the foundation rock and the possibility of cracking in the grout cap creating high seepage gradients. If a grout cap is used, it generally is a concrete-filled trench excavated from 3 to 8 feet into the bedrock, depending on the extent of broken rock; the trench is usually at least 3 feet wide to facilitate construction.

Grout pipes (nipples) are normally embedded at lo-foot centers in the foundation rock or grout cap, if used, during the concrete placement. Excavation for any grout cap must be carefully performed so that rock adjacent to the trench is not shattered.

Figure 6-12 shows the construction of a typical grout cap at Navajo Dam, New Mexico.

When grouting foundations in which the surface rock is broken or jointed, grout often rises to the surface through these cracks and prevents complete grouting. The cracks or seams through which grout rises to the surface should be caulked to prevent excessive leakage. Caulking can be done with wooden wedges, cement grout, or burlap. The grout pumped into the foundation may also be allowed to set within the cracks.

If it is highly probable that the foundation will require extensive grouting, a preliminary test program may be desirable. Such test programs furnish specific data with which the final grouting program may be carefully planned. Test grouting programs can eliminate expensive delays caused by large grout overruns and should expedite the completion of the job.

Specifications for the performance of foundation grouting and for the excavation of the grout cap are included in sections G.56 through G.60. If an extensive grouting program is contemplated, an engineer experienced in this type of work should be consulted.

At one time, concrete cutoff walls were constructed to intercept seepage along the contact of the embankment with the rock foundation. But these walls are expensive and prone to cracking, and their usefulness is questionable. They are not recommended for the earthfill dams discussed herein.

However, in unusual cases where the bedrock is very smooth, a cutoff wall may be warranted. In some very pervious rock foundations or those containing soluble zones or layers, such as limestone or gypsum, it may be appropriate to provide cutoffs through pervious zones to control seepage.

Cutoffs are also sometimes advisable through upper zones of weathered or broken foundation rock. Shallow cutoffs are usually provided by earth filled cutoffs with sloping sides. Where deep cutoffs are required, thin foundation cutoffs such as a concrete diaphragm wall may be more economical. USBR

Embankment Dams Design Standards No. 13, chapter 16, discusses foundation cutoff walls.

All loose and overhanging rock must be removed from the abutments; rock slopes should not be steeper than 0.5:1 (horizontal to vertical) and preferably flatter. Where flattening the rock slopes or overhangs is not practicable, the slopes may be shaped by the use of dental concrete.

If the bedrock is a shale that slakes in air, it may be necessary to excavate several feet into bedrock to remove the surface disintegration just before placement of the embankment; in more durable rock types, little excavation into the bedrock (other than for a grout cap) is usually necessary. Fractured rock should be treated by slush grouting (see sec. G.61). USBR Design

Standards No. 13, chapter 3, discusses foundation surface treatment in detail. A sample specification for construction on a shale foundation subject to slaking is included in appendix G.

In most instances, bedrock is mantled by overburden of various types and thicknesses. The foundation design then depends on the nature and depth of the overburden as described in succeeding section.

The above discussion is applicable not only to exposed rock foundations, but also to bedrock reached by trenching through the overburden.

Filters and drains are the most important features for collecting and controlling seepage through rock foundations. Even though a rock foundation may be grouted and cutoffs provided, appropriate filters and drainage are still necessary to collect seepage and reduce uplift pressures in the area downstream of the impervious zone. This is a necessary design measure that precludes unforeseen events such as foundation fracturing caused by earthquakes or construction deficiencies that may occur in grout curtains and cutoffs. Drainage blankets, toe drains, toe trenches, and relief wells should be used individually or in combination as necessary to control seepage. USER Design Standards No.13, chapters 5 and 8, cover the design of these features.

6.9. Sand and Gravel Foundations.-

(a) General.-Often the foundations for dams consist of recent alluvial deposits composed of relatively pervious sands and gravels overlying impervious geologic formations. The pervious materials may range from fine sand to openwork gravels, but more often they consist of stratified heterogeneous mixtures. Generally, sand and gravel foundations have sufficient strength to adequately support loads induced by the embankment and reservoir, but this must be verified by adequate exploration, testing, and analyses. Knowledge of the geologic deposition process can help determine the potential occurrence of low strength zones.

Two basic problems are found in pervious foundations; one pertains to the amount of under seepage, and the other is concerned with the forces exerted by the seepage. The type and extent of treatment justified to decrease the amount of seepage should be determined by the purpose of the dam, the stream flow yield in relation to the reservoir conservation capacity, and the necessity for making constant reservoir releases to serve senior water rights or to maintain a live stream for fish or for other conservation purposes. Loss of water through under seepage may be of economic concern for a storage dam but of little consequence for a detention dam. Economic studies of the value of the water and the cost of limiting the amount of under seepage are required in some instances to determine the extent of treatment. However, adequate measures must be taken to ensure the safety of the dam against failure caused by piping, regardless of the economic value of the seepage.

A special problem may exist in foundations consisting of low density sands and gravels. The loose structure of saturated sands and gravels is subject to collapse under the action of a dynamic load. Although the loose sand may support sizable static loads through point-to-point contact of the sand grains, a vibration or shock may cause the grains to try to readjust into a more dense structure. Because drainage cannot take place instantaneously, part of the static load formerly carried by the sand grains is then transferred temporarily to the water, and the effective strength of the foundation may be greatly reduced, possibly leading to failure. USER Design Standards No.13, chapter 13, covers seismic design and analyses.

Foundations consisting of cohesion less sand of low density are suspect, and special investigations should be made to determine required remedial treatment. If the relative density of the foundation is less than 50 percent, the approximate magnitude of the relative density of a cohesionless sand foundation can be determined from the results of standard penetration tests described in section 5.32(b).

The number of blows per foot is related to the relative density, but is affected by the depth of the test and, to some extent, by the location of the water table.

Special studies in tri-axial shear on undisturbed samples may be required for foundations of cohesion less sand indicated to be below 50 percent relative density. Such studies are beyond the scope of this text, and the advice of specialists in dam design should be obtained.

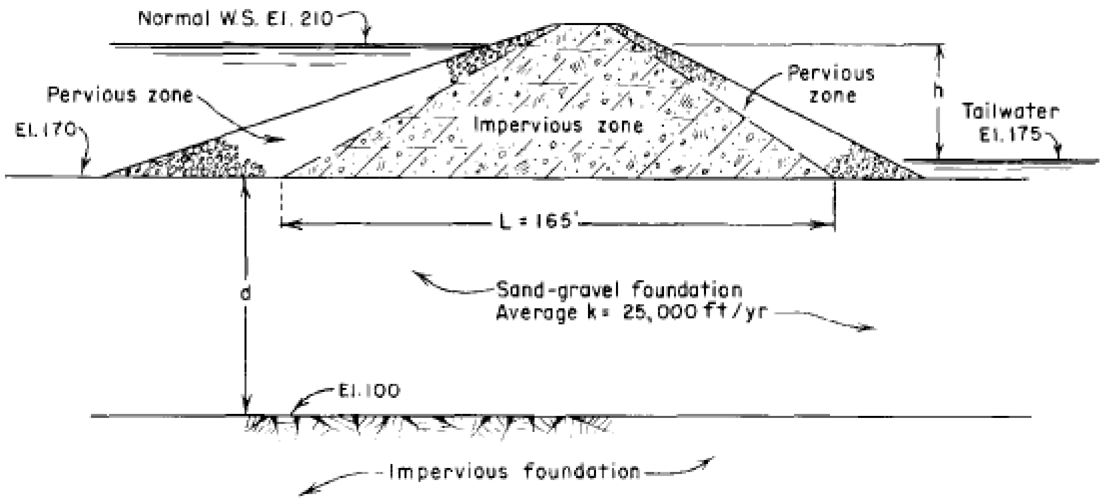


Figure 6-13.—Example computation of seepage by Darcy's formula. 288–D–2481.

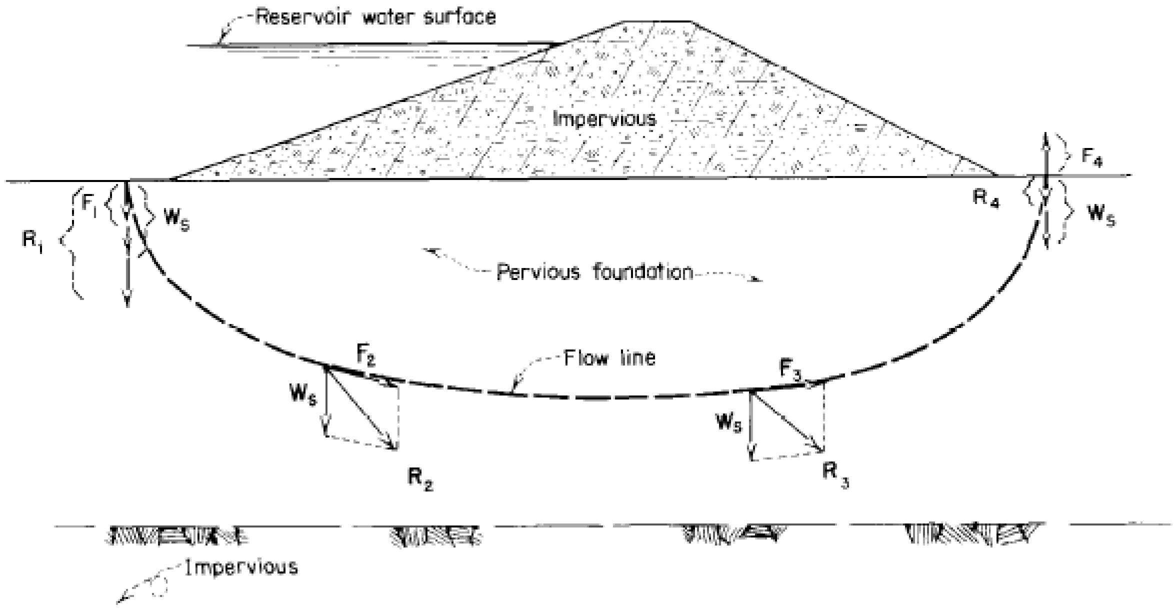


Figure 6-14.—Seepage force components. 288–D–2482.

6.10. Methods of Treating Sand and Gravel Foundations.-(a) General.-Various methods of seepage and percolation control can be used, depending on the requirements for preventing uneconomical loss of water and the nature of the foundation in regard to stability from seepage forces. Cutoff trenches, sheet piling, mixed-in-place concrete pile curtains, slurry trenches, grouting of alluvium, or combinations of these methods have been used to reduce the flow and to control seepage forces. Blankets of impervious material, extending upstream from the toe of the dam and possibly covering all or part of the abutments, are frequently used for the same purpose. Horizontal drainage blankets may be incorporated in the downstream toe of a dam or used to blanket the area immediately downstream from the toe of the dam through which percolating water may escape under an appreciable head. The purpose of these blankets is to permit free flow and dissipation of pressure without disruption of the foundation structure and loss of fine soil particles. Pressure-relief wells are used to relieve pressure in pervious layers or zones deeper in the foundation before the pressures are transmitted to the downstream toe area.

The details of these various devices together with an appraisal of effectiveness are contained in this section. The application of the various devices to the design of pervious foundations is included in section 6.11.

These various devices are

- Cutoff Trenches
- Partial Cutoff Trenches
- Sheet Piling Cutoffs
- Cement-Bound and Jet-Grouted Cutoff Trenches
- Slurry Trench Cutoffs
- Grouting
- Upstream Blankets
- Downstream Embankment Zones for pervious Foundations
- Toe Drains and Drainage Trenches
- Pressure Relief Wells

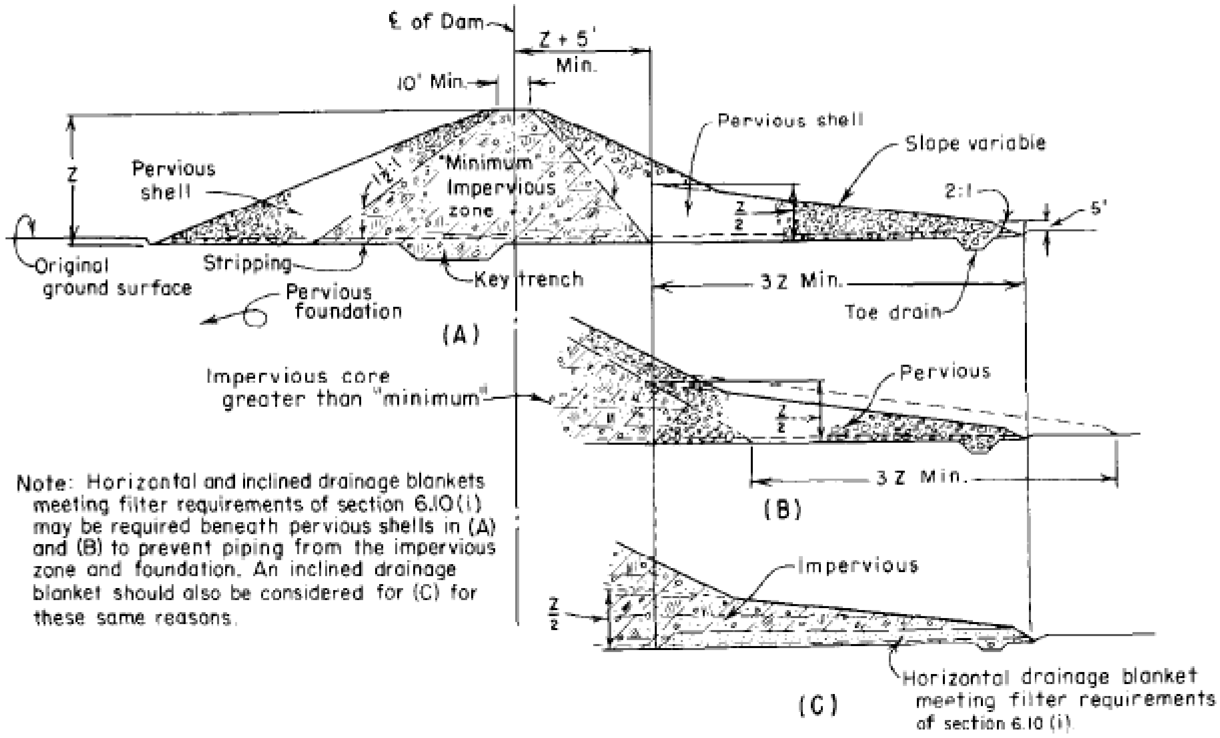


Figure 6-23.—Downstream embankment sections for pervious foundations. 288–D–2483.

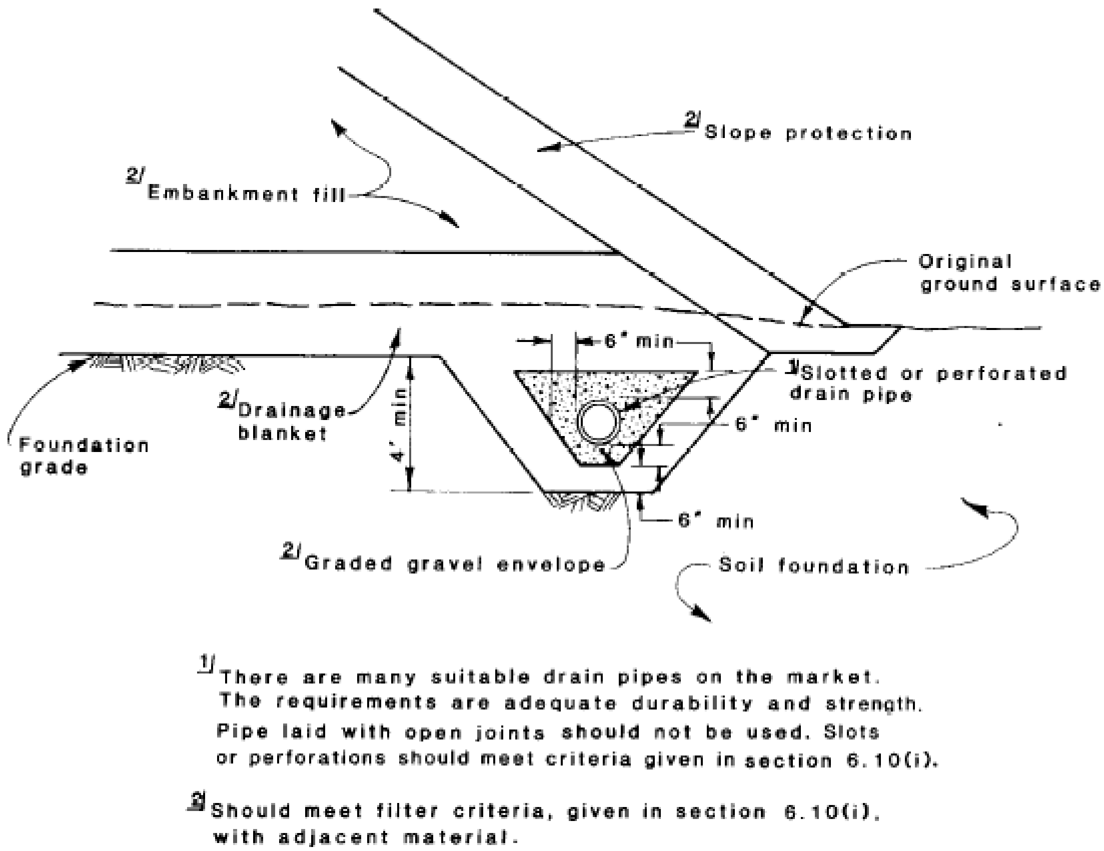
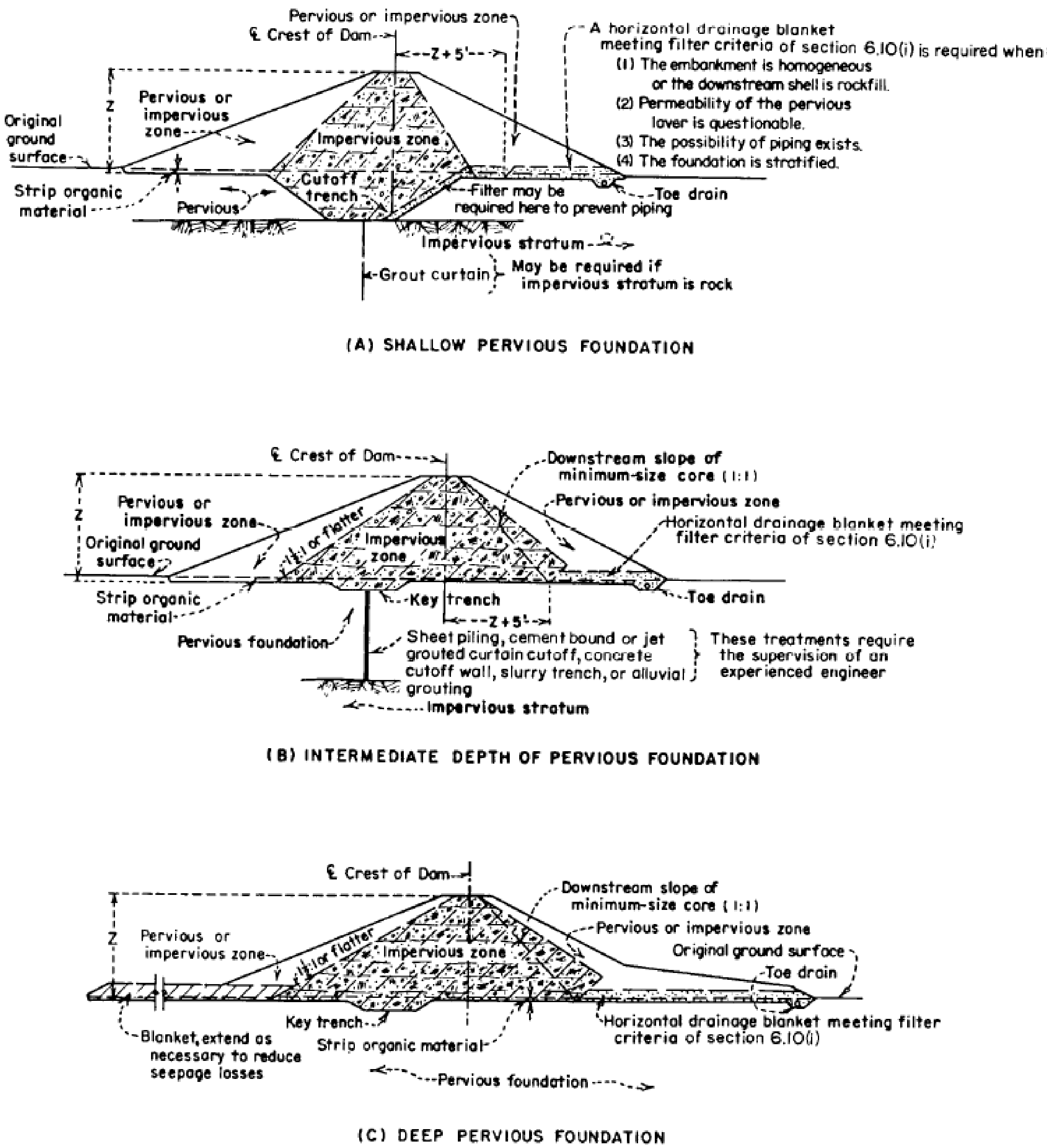


Figure 6-25.—Typical toe drain installation. 103–D–1829.

6.11. Designs for Sand and Gravel Foundations.-(a) General.-Criteria (b) for the design of earthfill dams, presented in section 6.5, require that the flow of seepage through the foundation and abutments be controlled so that no internal erosion occurs and there is no sloughing in the area where the seepage emerges. This criterion also requires that the amount of water lost through seepage be controlled so that it does not interfere with planned project functions. Section 6.6 discusses the basis used for designing foundations for small dams, which requires a generalization of the nature of the foundation in lieu of detailed explorations and the establishment of less theoretical design procedures than those used for major structures. Section 6.6 also cautions against the use of these design procedures for unusual conditions where procedures based largely on judgment and experience are not appropriate.

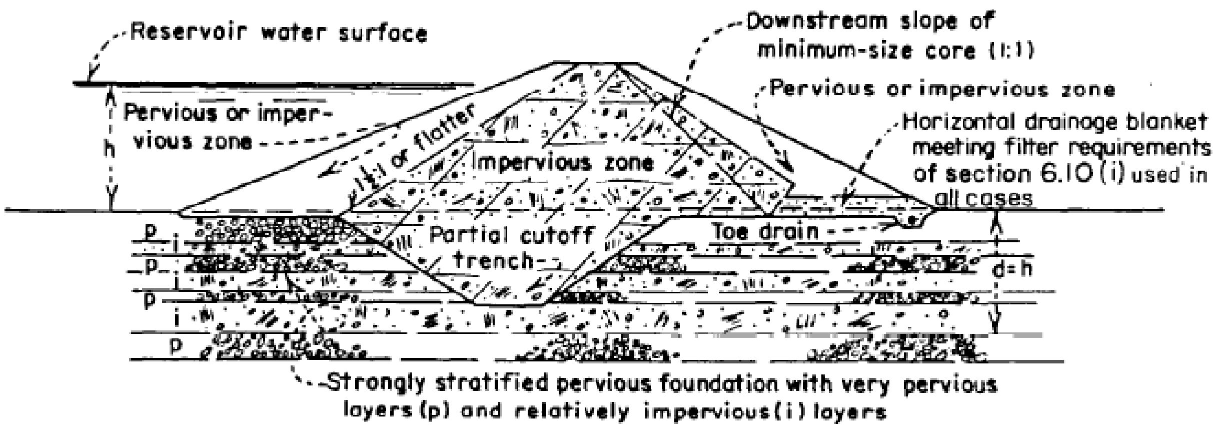
(f) Summary of Pervious Foundation Treatments. - Table 6-2 is a summary of recommended treatments for various pervious foundation conditions. Foundations are normally considered as either shallow or deep because these are by far the most common conditions encountered.

However, if the foundation is determined to be of intermediate depth, special construction methods are required that should be supervised by an experienced engineer. Intermediate depth foundations are discussed in section 6.11(c). The treatments of shallow and deep foundations, both exposed and covered, are discussed in detail in sections 6.11(d) and (e).



NOTE: Filter criteria given in section 6.10(i) applies between the impervious zone and any downsteam zone or a properly designed filter must be provided on (A),(B) and (C).

Figure 6-28.—Treatment of Case 1: exposed pervious foundations. 288–D–2486.



Notes: (1) If stratified foundation is of shallow depth, a positive cutoff trench should be used.
(2) Pressure relief wells may be required for deeply stratified foundations.
(3) Filter criteria given in section 6.10(i) applies between the impervious zone and any downstream zone or foundation layer, otherwise a filter should be provided.

Figure 6-29.—Treatment of stratified foundations. 288–D–2487.

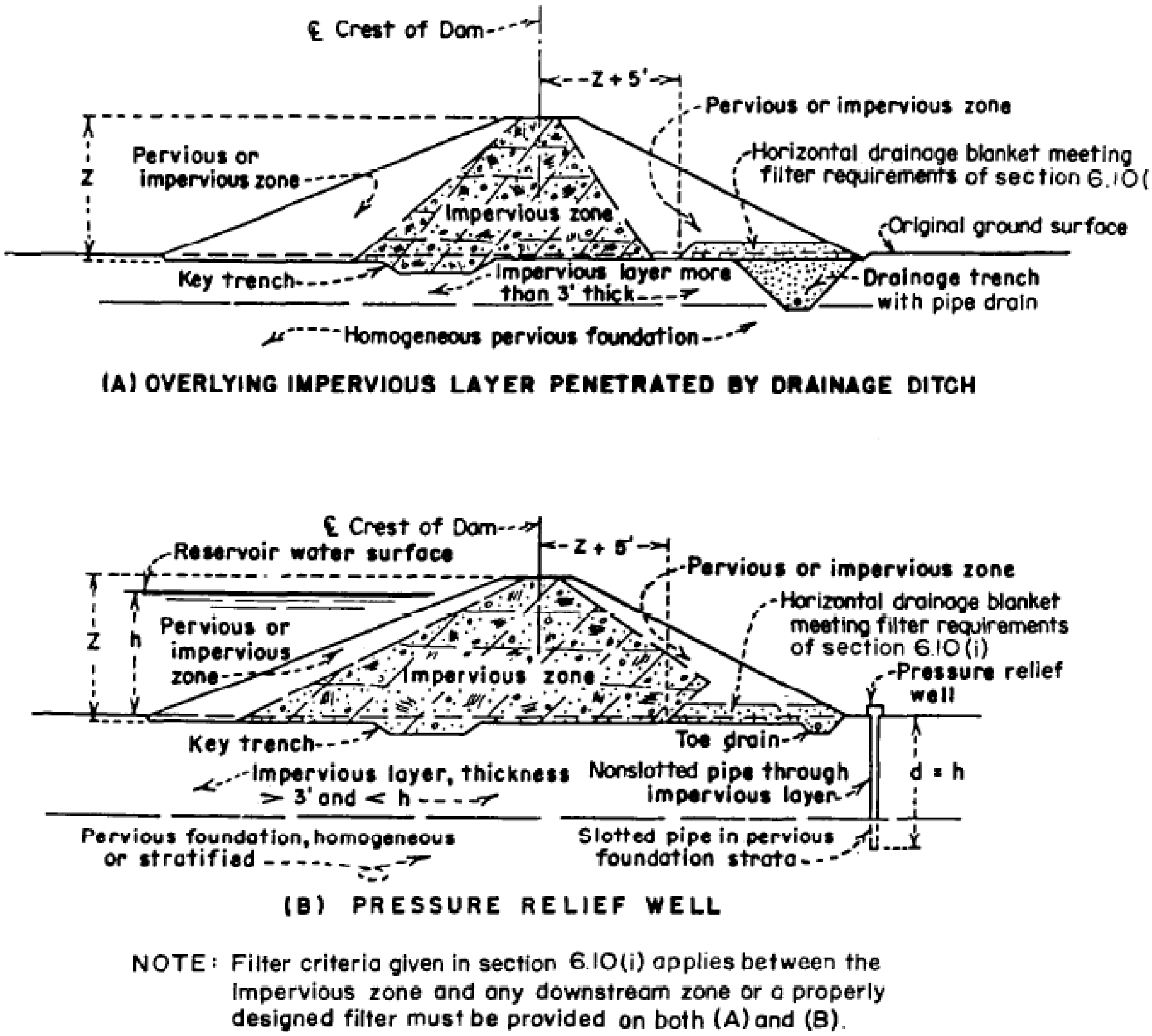


Figure 6-30.—Treatment of Case 2: covered previous foundations. With overlying impervious layer of thickness more than 3 feet but less than the reservoir head. 288–D–2488.

6.12. Methods of Treating Silt and Clay Foundations.-(a) General.-Foundations of fine grained soils are usually impermeable enough to preclude the necessity of providing design features for under seepage and piping. However, as discussed previously, inclined and horizontal filter-drainage blankets provide good protection against unknown geologic conditions, cracking, dispersive soils, and design and construction defects. Purely homogeneous dams are no longer recommended except for the most unimportant structures. Filter-drainage blankets should meet the criteria in section 6.10(i).

The main problem with these foundations is stability. In addition to the obvious danger of bearing failure of foundations of saturated silts and clays, the designs must take into account the effect of foundation saturation of the dam and of appurtenant works by the reservoir.

Methods of foundation treatment are based on the soil type, the location of the water table, and the density of the soil. For saturated foundations of fine-grained soils (including sands containing sufficient fines to make the material impervious), the standard penetration test described in section 5.32(b) provides an approximate measure of the density or relative consistency. This test cannot be relied on, however, in fine-grained soils above the water table, especially very dry soils whose resistance to penetration is high although their unit weight is low. In these soils, the unit weight can be determined by in place unit weight tests described in section 5.47.

6.13. Designs for Silt and Clay foundations. - (a) Saturated Foundations.-The designs of small dams on saturated fine-grained soils given in this section are based on the results of numerous stability analyses using various heights of dam and different sets of slopes for the stabilizing fills for each height. Average values of embankment properties were used and the required shearing strength for a safety factor of 1.5 was determined assuming that no drainage occurred in the foundation during construction.

This construction condition was found to be more severe for stability than either the steady state seepage condition or the sudden drawdown condition. Furthermore, the type of material used for embankment and stabilizing fills was found to have no appreciable effect on the stability, which was a function of the soil type and the relative consistency of the saturated foundation. The slopes of stabilizing fills were determined by finding the various combinations of cohesion and tan ϕ of the foundation soil needed to provide a 1.5 safety factor for the critical condition using the Swedish slip circle method,

Figure 6-33 shows a typical section design for a small dam on a saturated fine-grained foundation.

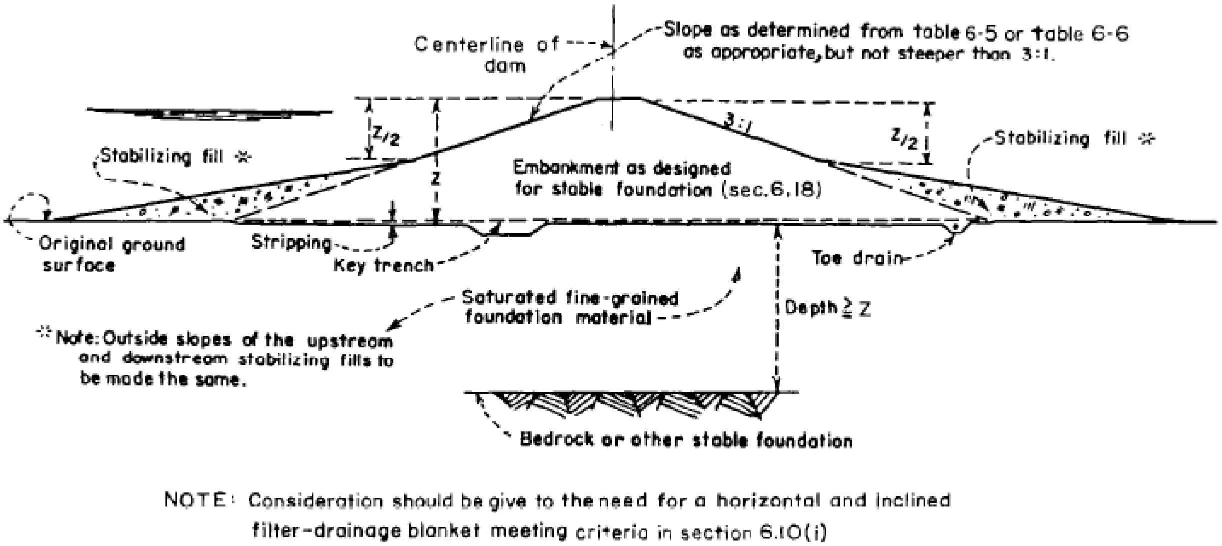


Figure 6-33.—Design of dam on saturated fine-grained foundation. 288–D–2491.

Table 6-3 lists the recommended slopes for stabilizing fills for saturated foundations typical of the groups of the Unified Soil Classification System for different degrees of consistency. Blows per foot of the standard penetration test are used to approximate relative consistency: Less than 4 blows corresponds to G, = 0.50,4 to 10 blows corresponds to G, = 0.5 to 0.75, 11 to 20 blows corresponds to G, = 0.75 to 1.0, and more than 20 blows corresponds to G, = 1.0. Recommendations are not made for slopes of soils averaging less than four blows per foot within a foundation depth equal to the height of the dam. These very soft foundations require special sampling and testing methods that are beyond the scope of this text.

Example

Required:

Slope of stabilizing fill for a safety factor of approximately

1.5.

Given:

Type of dam = either homogeneous or zoned.

Foundation blow count from field tests = 15.

Saturated foundation material = CL.

Height of dam = 40 feet.

Solution:

From table 6-3 opposite stiff consistency and CL, read 4.5:1 under dam height of 40 feet.

D. EMBANKMENTS

6.14. Fundamental Considerations.-Essentially, designing an earthfill dam embankment primarily involves determining the cross section that, when constructed with the available materials, will fulfill its required function with adequate safety at a minimum cost. The designer of an earthfill dam cannot rely on the application of mathematical analyses or formulas to determine the required cross section to the same degree that one can for a concrete dam. Soils occur in infinite combinations of size gradation, composition, and corresponding variations in behavior under different conditions of saturation and loading. In addition, the stress-strain relationships in a soil embankment are very complex.

Considerable progress has been made in investigations and studies directed toward the development of methods that will afford a comprehensive analysis of embankment stability. These methods provide useful design tools, especially for major structures where the cost of detailed explorations and laboratory testing of available construction materials can be justified on the basis of savings achieved through precise design. Even so, present practice in determining the required cross section of an earthfill dam consists largely of designing to the slopes and characteristics of existing successful dams, making analytical and experimental studies for unusual conditions, and controlling closely the selection and placement of embankment materials.

While some modifications are necessarily made in specific designs to adapt them to particular conditions, radical innovations are avoided and fundamental changes in design concepts are developed and adopted gradually through practical experience and trial.

Although the above practice may be criticized as being overly cautious and extravagant, no better method has been conclusively demonstrated. Where consideration is given to the possible loss of life, to the possibility of costly property damage, and to the waste of money incidental to the failure of a constructed dam, ample justification is provided for conservative procedures. For small dams, where the cost of explorations and laboratory testing of embankment materials for analytical studies together with the cost of the engineering constitutes an inordinate proportion of the total cost of the structure, the practice of designing on the basis of successful structures and past experience becomes even more appropriate.

The design criteria for earthfill dams are presented in section 6.5. In regard to the embankment, they require that the slopes of the embankment be stable under all conditions of construction and reservoir operation; that excessive stresses not be induced in the foundation; that seepage through the embankment be controlled; that the embankment be safe against overtopping; that the slopes be protected against erosion; and that the embankment be stable under appropriate seismic conditions.

This part of the chapter is concerned with the slope stability of the embankment under both static and seismic conditions and with the control of seepage through the embankment. Embankment details concerning the crest, freeboard, slope protection, and surface drainage are discussed in part E of this chapter.

The stability of an embankment is determined by its ability to resist shear stresses, which can cause failure by inducing sliding along a shear surface.

Shear stresses result from externally applied loads, such as reservoir and earthquake loads, and from internal forces caused by the weight of the soil and the embankment slopes. The external and internal forces also produce compressive stresses normal to potential sliding surfaces. These compressive stresses contribute both to the shear strength of the soil and to the development of destabilizing pore water pressures.

Granular, or non-cohesive, soils are more stable than cohesive soils because granular materials have a higher frictional resistance and because their greater permeability permits rapid dissipation of pore water pressures resulting from compressive forces. Accordingly, when other conditions permit, somewhat steeper slopes may be adopted for non-cohesive soils. Embankments of homogeneous materials of relatively low permeability have slopes generally flatter than those used for zoned embankments, which have free-draining outer zones supporting inner zones of relatively impervious materials.

In brief, the design of an earthfill dam cross section is controlled by the physical properties of the materials available for construction, by the character of the foundation, by the construction methods specified, and by the degree of construction control anticipated.

6.15. Pore Wafer Pressure.-In 1936, Terzaghi [39] demonstrated that in impervious soils subjected to load, a total stress normal to any plane is composed of an effective stress and a fluid pressure. The concepts of plane surfaces and stresses at a point in soils are not identical with those of an ideal homogeneous isotropic material. The “plane” in soils is a rather wavy surface, touching the soil particles only at their contacts with one another, and the “point” of stress is a small region containing enough of the particles to obtain an average stress.

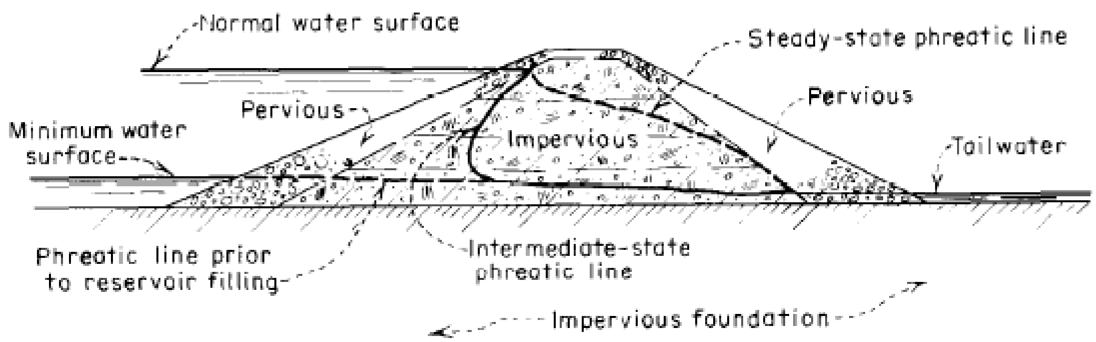


Figure 6-39.—Position of phreatic line in a zoned embankment. 288–D–2495.

6.16. Seepage Through Embankments.--The core, or water barrier portion, of an earthfill dam provides the resistance to seepage that contains the reservoir. Although soils vary greatly in permeability, as pointed out in section 5.18(c), even the tightest clays are porous and cannot prevent water from seeping through them.

The use of the flow net in determining the magnitude and distribution of seepage pressures in pervious foundations has been described previously (sec. 6.9(c)). The flow net can also be used to visualize the flow pattern of percolating water through embankments to estimate the magnitude and distribution of pressures from percolating water, both in the steady state and in the drawdown condition. Analytical methods of stability analyses used in the design of major structures require that such pore water pressures be determined quantitatively. Such a determination is not required for the design procedure given in this text.

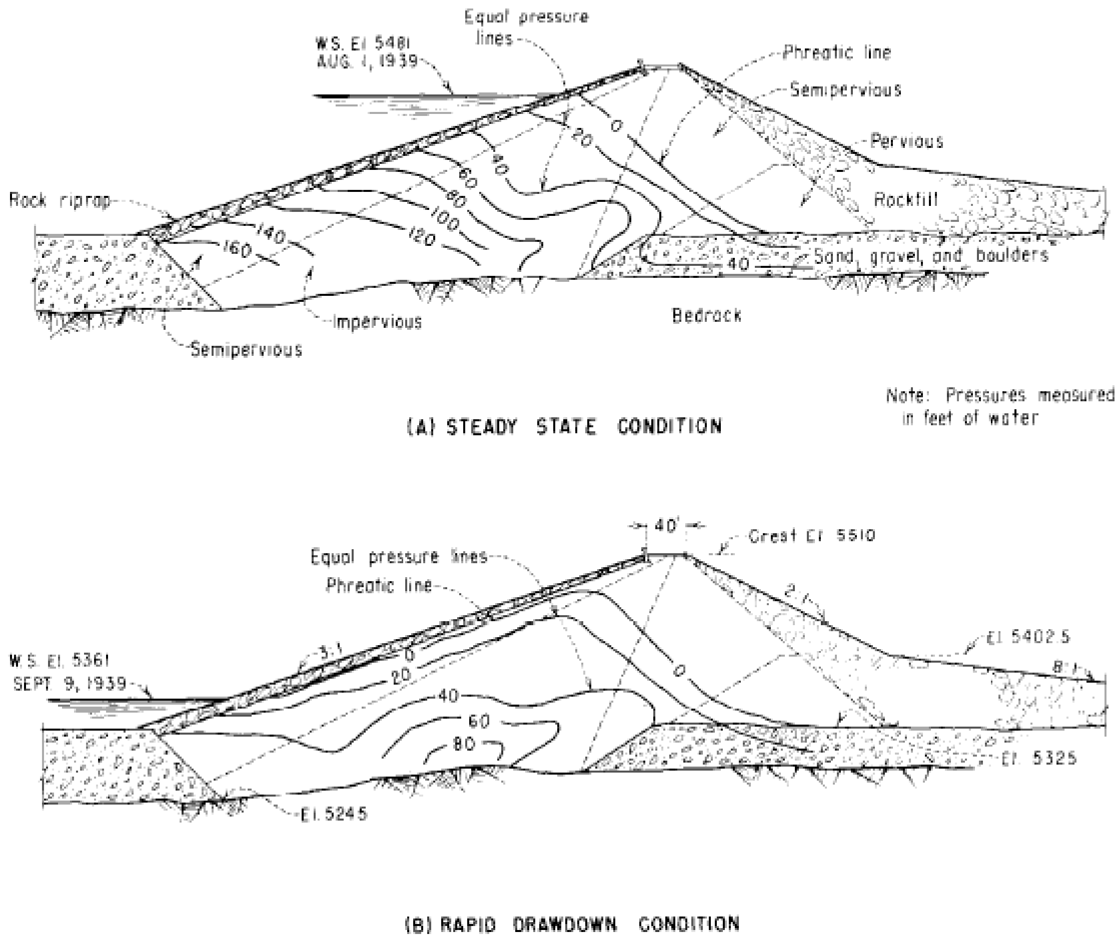


Figure 6-40.—Effect of rapid drawdown on pore pressures. Alcova Dam, an earthfill structure on the North Platte River in Wyoming. 288–D–2496.

6.17. Stability Analyses.-Various methods have been proposed for computing the stability of earthfill dams [6]. In general, these methods are based on the shear strength of the soil and certain assumptions with respect to the character of an embankment failure. The Swedish, or slip-circle, method, which supposes the surface of rupture to be a cylindrical surface, is a comparatively simple method of analyzing embankment stability. Although other more strictly mathematical solutions have been developed, the slip-circle method of stability analysis is generally adequate for small dams.

In this method, the factor of safety against sliding is defined as the ratio of the average shear strength, as determined from equation (lo), to the average shear stress determined by statics on a potential sliding surface. If there are weak lines or segments, such as weak foundation layers, failure surfaces involving these segments should be checked.

The force exerted by any segment within the slip circle is equal to the weight of the segment and acts vertically downward through its center of gravity.

The components of this weight acting on a portion of the circle are the force normal to the arc and the force tangent to the arc, as determined by completing the force triangle with lines in the radial and tangential directions. Pore water pressures acting on the arc result in an uplift force, which reduces the normal component of the weight of the segment. Graphical means have been developed by May [42] to facilitate the solution.

6.18. Embankment Design.-(a) Use of Materials from Structural Excavation.--In the discussion of design criteria (sec. 6.5), it was pointed out that for minimum cost, the dam must be designed to make maximum use of the most economical materials available, including the materials excavated for the dam foundation, spillway, outlet works, canals, powerhouse, roadways, and other appurtenant structures. When the yardage from these sources constitutes an appreciable portion of the total embankment volume, it may strongly influence the design of the dam. Although these materials are often less desirable than soil from available borrow areas, economy requires that they be used to the maximum practicable extent. Available borrow areas and structural excavations must both be considered to arrive at a suitable design.

(b) Embankment Slopes, General.-The design slopes of an embankment may vary significantly depending on the character of the materials available for construction, foundation conditions, and the height of the structure. The embankment slopes, as determined in this section, are the slopes required for stability of the embankment on a stable foundation. For stability against seepage forces, pervious foundations may require the addition of upstream blankets to reduce the amount of seepage or the addition of downstream inclined and horizontal filter-drainage blankets. Weak foundations may require the addition of stabilizing fills at either or both toes of the dam. The additional embankments needed because of pervious or weak foundations should be provided beyond the slopes determined herein as required for embankment stability.

The following procedures should be used with simple, straightforward geologic conditions and trouble-free embankment materials. If more complicated conditions exist, the dam should be analyzed by an experienced embankment dam designer using appropriate analytical techniques.

(c) Diaphragm Type.-A diaphragm dam consists of a thin impervious water barrier used in conjunction with a large pervious zone. The diaphragm can be constructed of earth, asphalt, concrete, or metal. If the diaphragm is constructed of impervious earth material, it must have a horizontal thickness at least great enough to accommodate construction equipment. Because it must hold back the full reservoir pressure, it must be constructed carefully. To prevent piping or erosion, the diaphragm must be protected by graded filters meeting criteria listed in section 6:10(i). When an earth diaphragm is centrally located, it is also referred to as a thin core. An earth diaphragm constructed for Amarillo Regulating Reservoir is shown on figure 6-58.

(d) Homogeneous Type.-The homogeneous type dam is recommended only where the lack of free-draining materials make the construction of a zoned embankment uneconomical, with the further qualification that for storage dams the homogeneous dam must be modified to include internal drainage facilities. The recommended drainage facilities for modified homogeneous dams are described in section 6.3 and are shown on figure 6-5. If a rockfill toe is provided, a filter must be constructed between the embankment proper and the rockfill toe, as shown on figure 6-5(A). This filter and the horizontal and inclined drainage blanket shown on figure 6-5(B) and 6-5(C) should be designed to meet the filter requirements described in section 6.10(i).

Table 6-5.—Recommended slopes for small homogeneous earthfill dams on stable foundations.

Case	Type	Purpose	Subject to rapid drawdown[1]	Soil classification[2]	Upstream slope	Downstream slope
A	Homogeneous or modified-homogeneous	Detention or storage	No	GW,GP,SW,SP	Pervious, unsuitable	
				GC,GM,SC,SM	2.5:1	2:1
				CL,ML	3:1	2.5:1
				CH,MH	3.5:1	2.5:1
B	Modified-homogeneous	Storage	Yes	GW,GP,SW,SP	Pervious, unsuitable	
				GC,GM,SC,SM	3:1	2:1
				CL,ML	3.5:1	2.5:1
				CH,MH	4:1	2.5:1

[1]Drawdown rates of 6 inches or more per day after prolonged storage at high reservoir levels.
[2]OL and OH soils are not recommended for major portions of homogeneous earthfill dams. Pt soils are unsuitable.

(e) Zoned Embankments.-(l) General.-The zoned embankment dam consists of a central impervious core flanked by zones of material that are considerably more pervious. An excellent example of a zoned dam from the 1950 era is Carter Lake Dam No. 3 (fig. 6-63). An excellent example from a more recent era is Ute Dam Dike (fig. 6-86). This type of embankment should always be constructed where there are a variety of soils readily available because its inherent advantages lead to savings in the costs of construction. Three major advantages in using zoned embankments are listed below.

- Steeper slopes may be used with consequent reduction in total volume of embankment materials
- A wide variety of materials may be used
- Maximum use can be made of material excavated from the foundation, spillway, outlet works, and other appurtenant structures.

(2) Zoning.-All zoning schemes are based on the estimated quantities of required excavation and of borrow area materials available. The zoning scheme may divide the dam into two or more zones, depending on the range of variation in the character and gradation of the materials available for construction. In general, the permeability of each zone should increase toward the outer slopes.

(3) Transitions.-It is important that the gradation of adjacent zones be considered so that materials from one zone are not “piped” into the voids of adjoining zones, either by steady-state or by drawdown seepage forces. Transitions prevent piping and also provide the additional advantage that should the embankment crack, partial sealing of the cracks takes place with subsequent reduction in seepage losses.

(4) Impervious Cores for Zoned Embankment- Figure 6-43 shows the suggested size of the minimum core for the following two conditions:

- Impervious or pervious foundations of shallow depth penetrated by a positive cutoff trench. This core is hereinafter referred to as minimum core A.
- Exposed pervious foundations and covered pervious foundations (<3 ft of cover) not penetrated by a positive cutoff trench regardless of the depth of pervious material. This core is hereinafter referred to as minimum core B.

(5) Embankment Slopes.-Table 6-6 shows the recommended upstream and downstream slopes for small zoned earthfill dams with minimum core A and with maximum core. The assumption is made that the foundation is stable; if the foundation is of the saturated fine-grained type, stabilizing fills, as described in section 6.13, should be added. Slopes of small zoned earthfill dams with cores of intermediate size (including minimum core B for dams on pervious foundations) fall between those given in the table for minimum core A and for the maximum size core.

Where only one slope is shown for more than one soil classification, it indicates that the embankment can be constructed using any of the soils or any combination thereof.

The following example illustrates the procedure:

Example

Required:

Upstream and downstream slopes for a zoned earthfill storage dam, 50 feet high, on a stable foundation subject to rapid drawdown.

Given:

Foundation = shallow, exposed, and pervious.

Shell material = SW and SP, both gravelly.

Core material = CL.

Solution:

Because the foundation is shallow and a positive cutoff trench can be constructed, minimum core A should be used. From table 6-6, select upstream and downstream slopes of 2:l.

6.19. Seismic Design.-The design and construction practices for small earth dams presented herein are considered adequate in areas of low seismicity, and the safety factors used should preclude major damage for all but the most catastrophic earthquakes.

Although all dam sites are subject to earthquake activity, the probability of an earthquake is greater in some regions than in others. This probability is generally determined by the number of previous earthquakes in that region and their intensity. In some cases, maps have been prepared that delineate certain areas having greater earthquake potential. One such seismic risk map is shown on figure 6-44. This map, adapted from Algermissen [47], uses the information collected and abstracted for approximately 28,000 earthquakes in the conterminous United States. If the designer is uncertain about the prospects of an earthquake in any area, a competent geologist or seismologist should be consulted.

After determining that the region is subject to earthquakes, the dam site should be inspected by an experienced engineering geologist to determine whether faults or detrimental geologic formations could affect the location of the dam, reservoir, or appurtenant structures. If active faults, unstable alluvial foundations, or the possibility of massive landslides into the reservoir exist, the dam site should be relocated. In general, the designer should assume that a dam within a seismic zone will be shaken by an earthquake.

If foundations consisting of low relative density sands and silts (sec. 6.9) or uniform, fine-grained, cohesion less materials are encountered, serious damage may result to the structure during an earthquake, and the assistance of an experienced dam designer is required. If an active fault exists at the proposed dam site, the designs proposed herein are inadequate.

Table 6-6.—Recommended slopes for small zoned earthfill dams on stable foundations.

Type	Purpose	Subject to rapid drawdown[2]	Shell material classification	Core material classification[3]	Upstream slope	Downstream slope
Zoned with minimum core A[1]	Any	Not critical[4]	Rockfill, GW, GP, SW (gravelly), or SP (gravelly)	GC, GM, SC, SM, CL, ML, CH, or MH	2:1	2:1
Zoned with maximum core[1]	Detention or storage	No	Rockfill, GW, GP, SW (gravelly), or SP (gravelly)	GC, GM SC, SM CL, ML CH, MH	2:1 2.25:1 2.5:1 3:1	2:1 2.25:1 2.5:1 3:1
Zoned with maximum core[1]	Storage	Yes	Rockfill, GW, GP, SW (gravelly), or SP (gravelly)	GC, GM SC, SM CL, ML CH, MH	2.5:1 2.5:1 3:1 3.5:1	2:1 2.25:1 2.5:1 3:1

[1]Minimum and maximum size cores are as shown on figure 6-43.
[2]Rapid drawdown is 6 inches or more per day after prolonged storage at high reservoir levels.
[3]OL and OH soils are not recommended for major portions of the cores of earthfill dams. Pt soils are unsuitable.
[4]Rapid drawdown will not affect the upstream slope of a zoned embankment that has a large upstream pervious shell.

E. EMBANKMENT DETAILS

6.20. Crest Design.-(a) General.-In designing the crest of an earthfill dam, the following items should be considered:

- Width
- Drainage
- Camber Surfacing
- Safety requirements
- Zoning

In addition, suitable parking areas should be provided at the abutments of the dam for the convenience of visitors and others, especially for a storage dam whose lake will be used for recreational purposes.

A turnaround should be provided where vehicular traffic is permitted on a dam crest those dead ends at the opposite abutment.

(b) Width.-The crest width of depends on considerations such as (1) nature of embankment materials and minimum allowable percolation distance through the embankment at normal reservoir water level, (2) height and importance of structure, (3) possible roadway requirements, and (4) practicability of construction. The minimum crest width should provide a safe seepage gradient through the embankment at the level of the full reservoir. Because of practical difficulties in determining this factor, the crest width is, as a precedent.

The following formula is suggested for the determination of crest width for small earthfill dams:

$$w = z/5 + 10 \qquad (12)$$

where:

w = width of crest, in feet, and

z = height of dam, in feet, above the streambed.

For ease of construction with power equipment, the minimum width should be at least 12 feet. For many dams, the minimum crest width is determined by the requirements for the roadway over the dam.

(c) Drainage.-Surface drainage of the crest should be provided by a crown of at least 3 inches, or by sloping the crest to drain toward the upstream slope. The latter method is preferred unless the downstream slope is protected against erosion.

(d) Camber.-Camber is ordinarily provided along the crest of earthfill dams to ensure that the freeboard will not be diminished by foundation settlement or embankment consolidation. Selection of the amount of camber is necessarily somewhat arbitrary.

It is based on the amount of foundation settlement and embankment consolidation expected for the dam, with the objective of providing enough extra height so that some residual camber will remain after settlement and consolidation. This residual camber also improves the appearance of the crest placed on top of the crest for protection against damage by wave splash and spray, rainfall, wind, frost, and traffic when the crest is used as a roadway.

The usual treatment consists of placing a layer of selected fine rock or gravelly material at least 4 inches thick. If the crest constitutes a section of highway, the width of roadway and type of surfacing should conform to those of the highway.

(f) Safety Requirements.-When the crest of a dam is used as a highway, cable- or beam-type guardrails are usually constructed along both shoulders of the crest. If a highway crossing is not anticipated, the crest can be lined with guard posts at 25-foot intervals or, on very minor structures, by boulders placed at intervals along the crest. If little or no traffic will use the crest, treatment may not be necessary.

(g) Zoning. -Incorrect zoning of materials at the crest leads to poor construction control, lost time, and possibly local failure of the crest.

(h) Typical Crest Details.-Figure 6-84 and 6-85 show the crest detail for Wasco Dam: 6 inches of camber were provided and a minimum top width of 14 feet was maintained for the impervious zone to ensure adequate room for compaction by tamping rollers. The core material was placed 3.5 feet higher than the maximum water surface. Note also that the top of the core material is sloped toward the reservoir to facilitate drainage.

6.2 1. Freeboard.-Freeboard is the vertical distance between the crest of the embankment (without camber) and the reservoir water surface. The more specific term "normal freeboard" is defined as the difference in elevation between the crest of the dam and the normal reservoir water level as fixed by design requirements. The term "minimum freeboard" is defined as the difference in elevation between the crest of the dam and the maximum reservoir water surface that would result should the inflow design flood occur and the outlet works and spillway function as planned. The difference between normal and minimum freeboard represents the surcharge head (sec. (9.3).If the spillway is uncontrolled, there is always a surcharge head; if the spillway is gated, it is possible for the normal and minimum freeboards to be identical, in which case the surcharge head is zero.

Table 6-7.—Wave height versus fetch and wind velocity. From [55].

Fetch, mi	Wind velocity, mi/h	Wave height, ft
1	50	2.7
1	75	3.0
2.5	50	3.2
2.5	75	3.6
2.5	100	3.9
5	50	3.7
5	75	4.3
5	100	4.8
10	50	4.5
10	75	5.4
10	100	6.1

Table 6-8.—Fetch versus recommended normal and minimum freeboard.

Fetch, mi	Normal freeboard, ft	Minimum freeboard, ft
<1	4	3
1	5	4
2.5	6	5
5	8	6
10	10	7

6.22. Upstream Slope Protection.

(a) General.-The upstream slopes of earthfill dams must be protected against destructive wave action. In some instances, provision must be made against burrowing animals. The usual types of surface protection for usptream slopes are rock riprap, either dry-dumped or hand-placed, and concrete pavement. Other types of protection that have been used are steel facing, bituminous pavement, precast concrete blocks, soil-cement pavement, and (on small and relatively unimportant structures) wood and sacked concrete. The upstream slope protection should extend from the crest of the dam to a safe distance below minimum water level (usually several feet). In some cases, it is advantageous to terminate the slope protection on a supporting berm, but this is generally not required.

(b) Selecting the Type of Protection-

Experience has shown that in most cases, dumped riprap furnishes the best upstream slope protection at the lowest ultimate cost. Approximately 100 dams, located in various sections of the United States with a wide variety of climatic conditions and wave severity, were examined by the Corps of Engineers.

The results of this survey were used as a basis for establishing the most practical and economical means for slope protection [56]. The dams were from 5 to 50 years old and were constructed by various agencies. This survey found that:

1. Dumped riprap failed in 5 percent of the cases it was used; failures were due to improper size of stones.

2. Hand-placed riprap failed in 30 percent of the cases it was used; failures were due to the usual method of single-course construction.

3. Concrete pavement failed in 36 percent of the cases it was used; failures were generally due to inherent deficiencies with this type of construction.

In this chapter, the following types of slope protection will be discussed:

- Dumped rock riprap
- Hand-placed rock riprap
- Concrete pavement
- Soil-cement

6.23. Downstream Slope Protection.-If the downstream zone of an embankment consists of rock or cobble-fill, no special surface treatment of the slope is necessary. Downstream slopes of homogeneous dams or dams with outer sand and gravel zones should be protected against erosion caused by wind and rainfall runoff by a layer of rock, cobbles, or sod. Because of the uncertainty of obtaining adequate protection by vegetative cover at many dam sites, especially in arid regions, protection by cobbles or rock is preferred and should be used where the cost is not prohibitive. Layers 24 inches thick are easier to place, but a 12-inch-thick layer usually affords sufficient protection. If grasses are planted, only those suitable for the locality should be selected. Figure 6-56 shows the native grasses that have protected the downstream slope of the Bureau's Belle Fourche Dam from erosion for 50 years. Two drainage berms, one of which is shown in the photograph, are located on the downstream slope of this 115-foot-high dam. Usually, fertilizer and uniform sprinkling of the seeded areas is necessary to promote the germination and to foster the growth of grasses. Appendix G contains sample specifications for placing topsoil, planting seed, and watering the seeded area until completion of construction.

6.24. Surface Drainage.-The desirability of providing facilities to handle surface drainage on the abutments and valley floor is often overlooked in the design of earth-fill dams. The result is that, although the upstream and downstream slopes and the crest of the dam are protected against erosion, unsightly gullying occurs at the contact of the embankment with earth abutments from which vegetation has been removed during the construction operations. This gullying occurs especially when the abutments are steep.

Gullying most often develops along the contact of the downstream slope with the abutments. However, it can usually be controlled by constructing a gutter along the contact. The gutter may be formed from cobbles or rock used in the downstream surfacing; if the downstream slope is seeded, a concrete, asphalt, or dry-rock paved gutter should be provided. The likelihood of gullying of the abutments and gentle slopes of the valley floor caused by runoff from the downstream slope of the dam also should be considered; contour ditches or open drains may be needed to control erosion. Figure 6-57 shows typical sections of a contour ditch and an open drain.

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

6.25. Flared Slopes at Abutments.-If necessary, the upstream and downstream slopes of the embankment may be flared at the abutments to provide flatter slopes for stability or to control seepage through a longer contact of the impervious zone of the dam with the abutment. If the abutment is pervious and if a positive cutoff cannot be attained economically, it may be possible to obtain the effect of an upstream blanket by flaring the embankment.

The design of the transition from normal to flared slopes is governed largely by the topography of the site, the length of contact desired, and the desirability of making a gradual transition without abrupt changes both for ease of construction and for appearance.

Rock-fill Dams

A. GENERAL

7.1. Origin and Usage.-Rockfill dams are generally conceded to have had their origin over 100 years ago during the California Gold Rush. From the late 1800's to the mid-1930's, many rockfill dams were constructed. The design and construction of a number of these dams are described by Galloway [11]. Interest in constructing rockfill dams diminished after the mid-1930's because of the increased costs of obtaining and placing large amounts of rockfill material; although a number of large rockfill dams were constructed in the 1950's [2]. Rockfill dam construction has increased markedly since 1960.

This is attributed to the use of more remote sites, more economical quarrying and placing operations, the use of excavated material in random zones, better design details, more general knowledge concerning rockfills, and the recent advent of pumped storage projects in mountainous terrain. Recent progress in rockfill dams is discussed by Cooke [3]. The excellent performance of an increasing number of rockfill dams is another beneficial factor recommending their use.

Rockfill dams can prove economical when any of the following conditions exist:

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

Other factors that favor the use of a rockfill dam are the ability to place rockfill throughout the winter and the possibility of grouting the foundation while simultaneously placing the embankment. In addition, uplift pressures and erosion caused by seepage through rockfill material do not generally constitute significant design problems.

Increasing interest is being shown in using “flow through” rockfill sections in conjunction with diversion dams to handle sudden floodflows when the cost of diversion is high [4, 5, 6, 71. This type of structure requires that a grid of welded reinforcing rods be placed across the downstream face of the dam, below a given elevation, and anchored to the rockfill so that large flows through the embankment do not displace the rock. The rods are usually l/2 to 3/ inch in diameter and are spaced so that rectangles 1 foot vertically by 3 to 4 feet horizontally are blocked out on the downstream face of the embankment.

This grid is then welded to reinforcing rods that are anchored 10 to 15 feet in the rockfill.

The use of “flow through” rockfill dams presents the designer with unique problems concerning the extent of downstream reinforcing and the ability of the section to resist overtopping. Therefore, this type of structure should be designed only by an experienced design engineer.

7.2. Definition and Types of Rockfill Dams.- Rockfill dams have been defined as follows [8, 91:“A dam that relies ‘on rock, either dumped in lifts or compacted in layers,-as a major structural element.”

An impervious membrane is used as the water barrier and can be placed either within the embankment (internal membrane) or on the upstream slope (external membrane). Various materials have been used for this membrane including earth materials, concrete, steel, asphaltic concrete, and wood.

Rockfill dams may be classified into three groups, depending on the location of the membrane: (1) central core, (2) sloping core, and (3) upstream membrane, or “decked.” Each membrane location has its advantages and disadvantages, which varyaccording to the type of membrane, materials available at the site, and foundation conditions. Central and sloping cores, which are internal membranes, are generally constructed of impervious earth materials.

Economic analyses should be made to determine the type of material to use in constructing the membrane, whether it be internal or external.

If an internal membrane is selected, a central vertical core is recommended. This type of core provides maximum contact pressure with the foundation and requires less strict construction control than a sloping core.

If an external membrane is used, it should be constructed of concrete, asphaltic concrete, or steel.

Advantages of the internal membrane include (1) less total area exposed to water, (2) shorter grout curtain lengths, and (3) protection from the effects of weathering and external damage. The prime disadvantages of an internal membrane are the inability to place rockfill material without the simultaneous placement of impervious core material and filters, the inaccessibility of the membrane for damage inspection, the difficulty in correcting damage should it occur, and the dependence on a smaller section of the dam for stability against sliding. The difference in the abilities of central (internal) and upstream (external) membranes to distribute stabilizing reactions against sliding is shown on figure 7-l.

Upstream membranes have the following advantages:

- Readily available for inspection and repair.
- Can be constructed after completion of the rockfill section.
- Foundation grouting can be performed simultaneously with rockfill placement.
- A larger portion of the dam is available for stability against sliding.
- Can be used as slope protection.
- It is relatively easy to raise the dam at a later date.
- In wet climates, the absence of impervious soil fill simplifies and speeds construction.

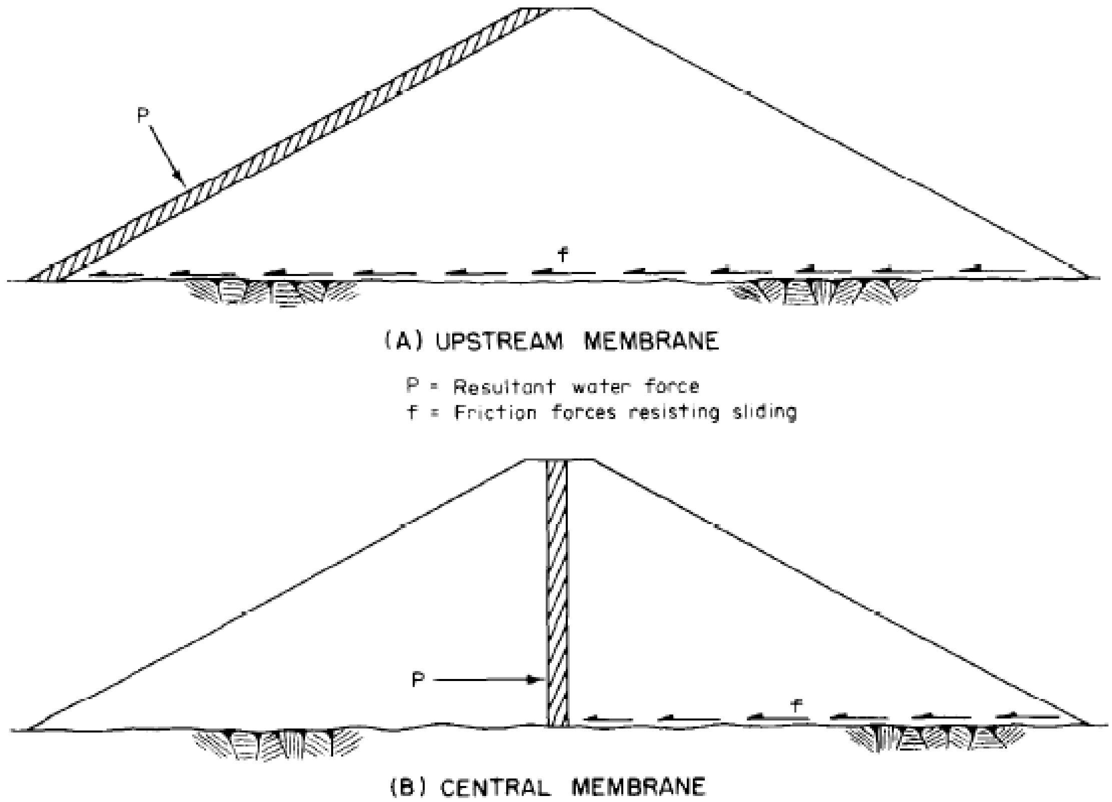


Figure 7-1.—Resistance to sliding for embankments. 288–D–2796.

If an upstream membrane is used, the reservoir should be capable of being drawn down to an elevation that will permit inspection and repair; television cameras or audio devices may be used for leak detection, and minor repairs may be made by divers.

If an earth-core rockfill dam is used, it requires the use of adequate filters both upstream and downstream; the filters should satisfy the requirements listed in section 6.10(i). If adequate earth material for either the core or the filter is not available at the site and separations of impervious material or manufactured filters are required, an earth-core rockfill dam may be uneconomical because filter processing costs can be extreme. Construction control costs of the earth-core rockfill dam will also be increased significantly if several filter layers are required to prevent piping.

B. FOUNDATION DESIGN

7.3. Foundation Requirements and Treatment.- The foundation requirements for a rockfill dam are less stringent than those for a concrete gravity dam, but more stringent than those for an earthfill dam.

Bedrock foundations that are hard and erosion resistant are the most desirable for rockfill dams.

Foundations consisting of river gravels or rock fragmentsare acceptable, but the foundation should be inspected by competent engineers and a positive cutoff to bedrock should be used. The foundation should be selected and treated from the viewpoint of providing minimum settlement to the rockfill embankment. All materials in cracks, faults, or deep pits that may eventually erode into the rockfill, either from the foundation or the abutment, should be covered with filters (sec. 6.10(i)) or removed and backfilled with concrete. For an earth-core rockfill dam, all joints and cracks beneath the core and the filters should be cleaned and filled with concrete [lO].

The usual method of treating the foundations to prevent under seepage is cement grouting beneath the cutoff; in addition, potential pervious zones upstream from the impervious membrane can be blanketed with impervious material.

The alignment of the dam should be selected so that either minimum embankment volume or minimum membrane exposure is attained, depending on which criterion is economically more important.

Foundation treatment must be sufficient to satisfy the following criteria:

- Minimum leakage
- Prevention of piping
- Limited settlement
- Sufficient friction development between abutments and foundation to ensure sliding stability

7.4. Membrane Cutoffs.-Of critical importance in the functioning of a rockfill dam are the prevention of seepage beneath the dam and the effecting of a watertight seal between the membrane and the foundation. To prevent seepage beneath the dam, foundations are usually grouted. The need for grouting and the extent required should be based on careful study of the site geology, on a visual examination of the drill cores from the rock foundation, and on drill-hole water-loss values. If no such data are available, it should be assumed that grouting will be required, except where the reservoir is completely drawn down each year and grouting requirements can be based on seepage observations during the first few years of operation, Cutoff walls excavated to various depths into bedrock are generally used to prevent leakage in the upper few feet of the foundation, to facilitate grouting operations, to provide a watertight seal with the membrane, and to support the downward thrust of the membrane. Figures 7-2, 7-3, and 7-4 illustrate typical cutoff wall details. Drainage galleries are sometimes used in conjunction with cutoff walls to facilitate later grouting and to determine seepage locations and quantities, but they are not recommended for small dams.

Recently [ll], designers have used the doweled cutoff slab shown on figure 7-5 in conjunction with concrete facings to provide the foundation membrane seal. Doweled cutoff slabs have the advantage of not requiring extensive excavations in rock, thereby allowing grouting operations to begin earlier, speeding completion time, and reducing design costs. Doweled cutoff slabs can be used where the bedrock is sound and few under seepage- problems are expected. When uncertainty concerning the permeability of upper portions of the foundation contact exists, such as for soft rock, a cutoff wall into bedrock can provide increased protection and allow an examination of questionable material.

A minimum width and depth of 3 feet is recommended for cutoff walls in sound rock; deeper walls should be used in unsound, broken, or closely jointed rock. The width of the doweled slab should be determined by foundation, construction, or grouting requirements. In addition to preventing under seepage, both the cutoff wall and the doweled cutoff slab must be designed to provide adequate support for the thrust of the membrane and, in the case of steel membranes, any tension imparted to the cutoff caused by embankment settlement.

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

C. EMBANKMENT DESIGN

7.5. Selection of Rock Materials.-A great variety of rock types have been used in the construction of rockfill dams. The types of rock have ranged from hard, durable, granite and quartzite to weaker materials, such as graywacke sandstone and slaty shale. In earlier years, it was thought that only the highest quality rockfill material should be used; however, with the advent of thinner lifts and more efficient compaction techniques, rock having less desirable characteristics has become feasible for use within embankment sections. The use of rock from excavations for spillways, outlet works, tunnels, and other appurtenant structures has reduced the construction costs of rockfill dams without impairing the usefulness or stability of embankments. If small amounts of the less desirable rock types are available, they can be used in random zones within the embankment; the use of material in random zones is discussed in section 6.18(a).

Preferably, rock material should be hard, durable, able to withstand disintegration from weathering, and able to resist excessive breakdown from quarrying, loading, hauling, and placing operations.

(Figure 7-6 shows the granite rockfill on the downstream face of Montgomery Dam.) The rock should also be free of unstable minerals that would weather mechanically or chemically, causing the rock to disintegrate.

Igneous, metamorphic, and sedimentary rocks have all been used successfully in embankment sections, and only general advice can be given concerning rock types because each dam site presents a unique problem in the use of the nearby rock materials. As an aid to the designer, part E of chapter 5 gives the classification and engineering properties of rocks.

Results from laboratory tests that measure the abrasion resistance, freeze-thaw characteristics, and the percent of water absorption can be used to evaluate the suitability of the rockfill. Results from petrographic analyses can be used to distinguish minerals known to weather easily, and unconfined or tri-axial compression tests can determine the strength properties of the rock. One of the best methods of determining the resistance of a rock to weathering is simply to examine its in place condition; however, this does not always indicate how the material will perform within the fill. Materials available at the site should be examined by constructing test embankments if economically possible, especially when the material properties are questionable. Test fills can determine the following items:

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

As an example, Crisp [12] reports that significant changes were proposed in the design of Carters Dam from results obtained by testing embankment sections of quartzite, phyllite, and argillite.

The effect of quarry blasting methods on the gradation of the rock should also be examined, as well as the required extent of quarrying.

Also of great importance to the design engineer selecting the type of rock is the degree to which small-scale triaxial compression tests provide strength parameters applicable to the actual rockfill.

Very limited data are available on this subject; however, Leps [13] has summarized available data and Marachi et al. [14] have examined this problem by testing 36-, 12-, and 2.8-inch-diameter specimens in drained triaxial compression tests using parallel grain-size curves and identical grain slopes (modeling) to examine the effects of grain size on the strength and deformation characteristics of rockfill material. They also investigated the effect of particle crushing.

Three types of material were tested as follows:

1. Argillite from Pyramid Dam. -A fine-grained sedimentary rock, quarry blasted, angular, with relatively weak particles (GB = 2.67)

2. Crushed basalt. -Quarry blasted and crushed to the correct size, angular, and quite sound (G. = 2.87).

3. Amphibolite from Oroville Dam. -A meta-volcanic rock, rounded to sub-rounded particles with some subangular fine-sand particles, river-dredged material, hard (G. = 2.86 to 2.94).

The gradation curves for the actual rock fill material and for the modeled material are shown on figure 7-7.

Although [14] was primarily concerned with the use of rock fill material in high dams, the following general conclusions apply to rockfill dams of all sizes:

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

Figure 7 -8 illustrates the variation of the angle of internal friction with both particle size and confining pressure. Although most of the confining pressures shown are greater than those attained in small dams, the general reduction in friction angle shown on figure 7 -8 should be of great interest to designers.

The details of testing and further conclusions regarding the strength and deformation properties of rock fill materials and the crushing characteristics of rock subjected to high confining pressures can be found in [14].

7.6. Embankment Sections.-Embankment slopes used for rockfill dams have evolved from very steep slopes, usually 0.5:1 to 0.75:1 (horizontal to vertical) on early rock fill dams, to the flatter slopes of 1.3:1 to 1.7:1 used today. Earlier rockfill dams used upstream membranes exclusively and were constructed with steep upstream and downstream slopes to minimize the volume of rockfill. Because these slopes were considerably steeper than the natural slope of dumped rock, they were stabilized by thick zones of crane-placed, dry rubble masonry, which provided the bedding for the upstream slope protection. The rockfill portions of these dams were constructed by dumping the rockfill in thick lifts, which ranged from 30 to 165 feet. Later designs eliminated the rubble masonry on the downstream slope by flattening it to the angle of repose of the rock, but the very steep upstream slope was retained.

Because most of the upstream zones were constructed by crane placement of large rocks, the cost of the dams continually increased. Gradually, designers found that it was more economical to use slopes approximating the angle of repose of the rock material and to eliminate crane placement in favor of compacted rockfills.

The upstream and downstream slopes of a dam should be based on the type of impervious membrane and its location. Rockfill dams having central or sloping cores have slopes ranging from 2:l to 4:l upstream and downstream-usually tending toward 2:l or slightly steeper when all conditions are favorable.

However, dams with upstream membranes usually have upstream slopes of from 1.3:1 to 1.7:1 and downstream slopes approximating the natural slope of the rock.

Most asphaltic-concrete-faced dams have been constructed with upstream slopes of 1.6:l to 1.7:l to facilitate construction of the membrane; whereas, most steel- and concrete-faced rockfill dams have slopes of 1.3:l to 1.4:1. A review of available literature indicates that very few failures have occurred for these slopes. Therefore, small rockfill dams with good foundations could have 1.3:1

to 1.4:1 upstream slopes for concrete and steel membranes, and a 1.7:1 upstream slope for asphaltic-concrete facings. Downstream slopes of 1.3:1 to 1.4:1 may be used in all cases.

The upstream and downstream slopes for central or sloping earth-core rockfill dams depends on the size and soil properties of the earth core, the width of filter zones required, type of foundation material, drawdown requirements, construction sequence, etc., with each site presenting its own unique problems.

Generally, the upstream and downstream slopes of a typical earth-core rockfill dam are 2:l or slightly steeper where all conditions are favorable, but may be as flat as 4:l (or flatter) for unfavorable conditions. A typical embankment section for a central earth-core rockfill dam is shown on figure 7-9.

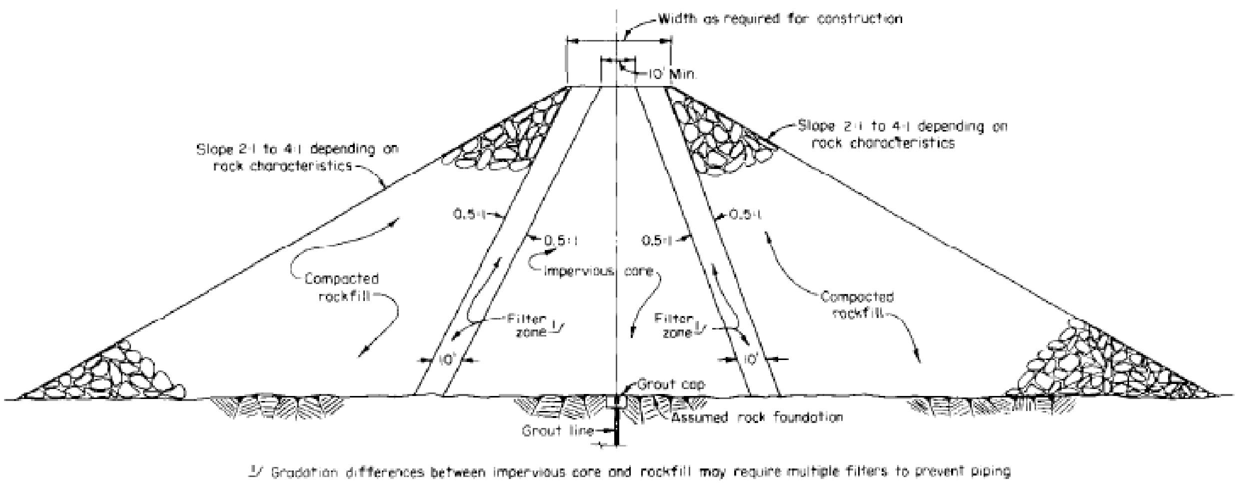


Figure 7-9.—Typical maximum section of an earth-core rockfill dam using a central core. 288–D–2800.

A typical section for a decked (upstream membrane) rockfill dam is shown on figure 7-10. The interior section of the decked rockflll dam can be divided into three major zones, as shown on figure 7-10. These zones can be described as follows:

- Zone C: The larger downstream zone of the dam, consisting of the best quality, larger, compacted rock; this zone provides high stability to the section.
- Zone B: Rock of lesser quality than zone C, such as that excavated from the spillway; used to minimize total dam costs.
- Zone A: Well-graded, smaller rock and gravel; used to provide bedding for the upstream membrane and to retard extreme water losses should the membrane crack.

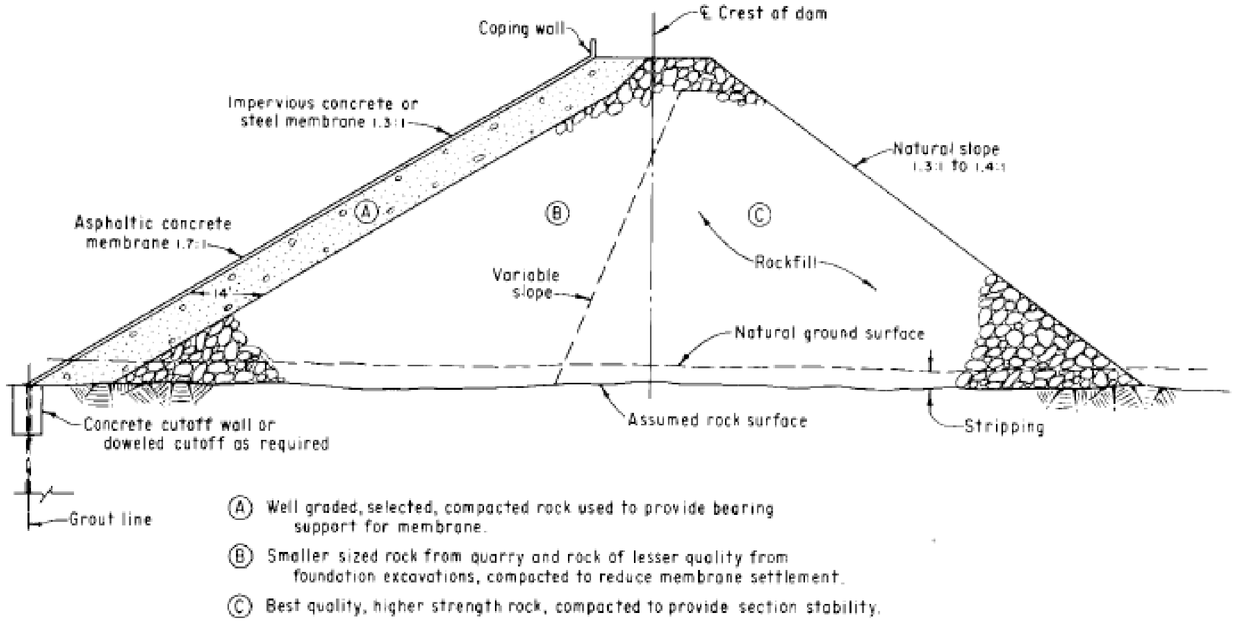


Figure 7-10.—Typical maximum section of a decked rockfill dam. 288–D–2801.

In addition to these major zones, a well-graded sand and gravel base course for the membrane is sometimes necessary. A thin base course also serves as a leveling course and provides a good working surface. Placement conditions for these three zones and the base course are discussed in section 7.7. Gradation requirements are difficult to specify because they depend on the type of rock available and the quarrying methods used. As with many aspects of dam design, only general rules apply, but filter requirements given in section 6.10(i) must be satisfied. For decked rockfill dams, zone C of the embankment should use the largest and best quality rock available. Large slabby rocks should not be placed in this fill because they tend to bridge, causing large voids that may result in excessive settlement should the rocks break. If possible, rock in zone C should be well-graded from approximately 1 ft3 to 1 yd3, and the finer fraction of the gradation should not be sufficient to fill the voids in the compacted material. Optimally, zone B should be wellgraded from a maximum size of about 10 ft3 and should have a high permeability after compaction.

Zone A should be well-graded from approximately 3 inches down to 5 to 15 percent passing the No. 100 sieve. If a base course is not necessary (as described later), the gradation of zone A depends on the type of facing used and its method of construction.

If a base course is not used, zone A material should provide a smooth uniform bearing surface for the facing, yet be graded to retard large water loss should the facing crack.

A base course may not always be necessary, depending on the need for a leveling course and on the gradation of zone A and its ability to withstand raveling during placement of the deck and to withstand erosion. In any event, the zone immediately beneath the face slab should provide a good working surface for equipment and workmen during placement of the facing element, and it should resist erosion during surface runoff. The base course material should be well-graded, with a maximum size of 1% inches, 5 to 15 percent passing the No. 100 sieve, and 5 percent or less passing the No. 200 sieve.

In general, material in zones B and C should grade from fine rock upstream to coarse rock downstream, with the largest and strongest material placed in the lower downstream portions of zone C.

Selection of the rock for each zone should be made at the quarry. For central earth-core rockfill dams, the larger and stronger rock should be placed in the outer rockfill zones. This rock should grade from fine rock next to the filter to coarse rock near the downstream slope.

The axis of the dam may be either curved (convex upstream) or straight. A curved axis allows the dam to be compressed as filling occurs; whereas, a straight axis has the benefit of easy construction layout and less total dam cost. For small dams with good foundation and abutment conditions, a straight axis is recommended. For the upstream membrane rockfill dams, it is also recommended that the layout be such that a minimum area of membrane face be exposed. This expedites face construction, reduces face and cutoff costs, and reduces the cost of any necessary repairs.

Random zones constructed of rock with questionable strength or permeability characteristics may also be used within the rockfill embankment if the stability of the section is not affected. The overriding purpose in the layout of any rockfill section is to make maximum economic use of the material available at the site. Test embankments can be used to determine whether or not materials will be adequate; these are discussed in section 7.5.

Crest width should be determined by the type of membrane used and by its use after construction.

The crest should, however, be wide enough to accommodate construction of the upstream membrane; a minimum width of 15 to 20 feet is recommended. Crest camber should be determined by the amount of foundation and embankment settlement anticipated. Because this is difficult to determine, a camber of 1 percent of the embankment height is recommended. A straight-line equation may be used to distribute the cambered material on the crest. Additional considerations concerning crest details are given in section 6.20.

Freeboard requirements depend on maximum wind velocity, fetch, reservoir operating conditions, spillway capacity, and whether coping walls are used. If a coping wall like that shown on figure 7-

10 is used to provide wave run up and over splash protection, the freeboard requirements of the embankment may be less than required for a riprapped earthfill dam (sec. 6.21). If a coping wall is not used, the freeboard should be adequate to prevent wave run up from flowing over the crest (sec. 6.21). Good results have been obtained with coping walls [ll], and their use is recommended.

7.7. Placement of Rockfill Materials.-Limiting settlement is critical in the construction of rockfill dams because excessive settlement may rupture the upstream membrane or cause joint separation with subsequent water loss. Early rockfill dams were constructed by dumping the rock in high lifts; it was assumed that dropping rock from heights imparted compaction energy to the fill, decreased the void space and, thus, reduced embankment settlement.

Nevertheless, many of these high lift embankments have since settled considerably with concurrent leakage problems. Experience has shown that rock material placed in thin layers and compacted by vibratory rollers forms a more stable mass in which settlement is minimal. For decked

rockfill dams, the embankment should preferably be completed before construction of the upstream membrane begins. This reduces the probability of serious membrane cracking by allowing initial settlement to occur.

Settlement of rockfill material has also been correlated with the application of water; Sowers et al. [15] have shown that rockfill material placed dry and subsequently wetted may settle appreciably.

Sluicing has long been advocated as a method that ensures that point-to-point bearing occurs between the larger rocks and that finer materials are washed into the voids. However, when rockfill material is placed in thin lifts and compacted by vibratory rollers, there appears to be no definite proof that sluicing operations significantly reduce the total settlement, especially for the smaller rockfill dams considered here.

The quantity of sluicing water used has varied extensively, but usually ranges from two to four times the volume of rock; dirty rock requires more water to wash the fines away. For exceptionally dirty rock, segregations may cause a water-saturated layer of fines to form below the surface of the rock as it is dumped over the edge of the lift and sluiced.

The layer will be relatively impermeable and will hinder or prevent wetting of all parts of the rock in the lift below the layer of fines. This may be corrected by using thicker lifts, which allows increased sluicing time, or possibly by wetting the rockfill before placement. Care should also be taken that mud does not form at the toe of the lift as a result of sluicing; if mud problems do arise, periodic removal should be mandatory. Sluicing is usually done with nozzles having diameters from 2% to 4 inches (a typical sluicing operation is shown on figure 7-U). Enough sluicing equipment should be available to handle maximum rock placing rates; otherwise, the quantity of rockfill placed may be limited. The sluicing equipment should be mobile.

Currently, placing rockfill in thin lifts and compacting it with a vibratory roller is the preferred construction method.

Figure 7-10 shows a typical decked rockfill dam section consisting of three zones of material. The zone C material should be sound, durable rock of high quality dumped in 2- to 4-foot lifts and compacted by a vibratory roller. Zone B material may be rock of lesser quality than that in zone C, such as spillway excavation or tunnel spoil, and should be dumped in 2- to 3-foot lifts and compacted by a vibratory roller. Zone A material provides the bearing surface for the upstream membrane and may be either a processed material or selected material from foundation or borrow pit excavations. Zone A material should be compacted to 12-inch lifts by either crawler-type tractors or vibratory rollers; the material should be thoroughly wetted before compaction.

The face of the zone A material should be compacted by drawing a smooth-drum vibratory roller up and down the face. Generally, the vibrator is turned off for the first two passes to prevent displacement.

If a base course is installed under the deck, it should also be compacted by drawing a smooth-drum vibratory roller up and down its face in the same manner described for the face of zone A. Suggested gradations for zones A, B, C, and base course material are discussed in section 7.6.

The size of the vibratory roller for each rockfill zone should be based on the properties of the rock in that zone and should, preferably, be established by constructing test embankments.

Vibratory rollers weighing from 3 to 10 tons have been the most widely used for rockfill compaction.

In a number of concrete- and asphalt-faced rockfill dams [11, 16, 17, 18], the zone A material has been eliminated and only a thin leveling course has been applied to the face of zone B. In such cases, compaction of the leveling course is performed by drawing a smooth-drum vibratory roller up its face.

Figure 7 -12 shows the maximum section of Upper Blue River Dam, in which zone A (zone 2 on the fig.) has been eliminated and zone B (zone 1 on the fig.) has been replaced by zone C. When zone A is eliminated, the final upstream surface of zone B may also be finished by pulling the vibratory roller up its face. The advice of an experienced dam designer should be obtained before zone A is eliminated.

For central earth-core rockfill dams, such as that shown on figure 7-9, the upstream and downstream rockfills should be compacted in 2- to 4-foot lifts by vibratory compactors to provide the most stable section possible. The fill should be thoroughly wetted to facilitate compaction. Sluicing operations used with this type dam require that great care be taken to ensure that filters are not clogged or impervious material washed away. The filter material should be compacted to 12-inch lifts either by crawler-type equipment or vibratory rollers. The width of the filter zones should be sufficient to accommodate placing and compacting equipment.

7.8. Seismic Design.-For areas of low seismic activity, the designs recommended herein should be adequate. The determination of potential earthquake activity within a given region can be made from a seismic risk map like that on figure 6-44 or by consultation with a seismologist or engineering geologist. If the dam site lies within a zone of high seismic activity, an experienced dam designer should be consulted.

It is the general opinion of many dam designers that large downstream zones of quarried rock compacted in thin lifts provide maximum stability against seismic shaking and maximum resistance to the flow of large quantities of water through the section should cracking occur. Thus, it is recommended that where seismic activity is expected, decked rockfill dams containing large downstream zones of compacted rockfill be used. The rockfill should, preferably, be well-graded, angular rock fragments of high strength and durability. To accommodate the larger downstream zones, it is recommended that where questionable earthquake conditions are encountered, the downstream slope of the decked rockfill be flattened to 1.7:1 in all cases. The upstream slope of the embankment should also be flattened if additional conservative design measures are warranted.

The foundation of the dam should, preferably, be firm rock; however, free-draining foundations (cobbles, boulders, rock fragments, etc.) may be used if their unit weight is similar to that of the rockfill material and they are approved by a competent dam designer. Trench-type foundation cutoffs are also recommended. In addition, it may be desirable to provide a thicker zone A (fig. 7-10) beneath the membrane, to require better quality rock for zone B, and to reduce the lift thickness to a maximum of 3 feet within zone C. Still another precaution would be using a thicker membrane on the upstream slope and, in the case of a concrete membrane, placing reinforcing in each face. It should be noted that there are no exact rules for design within earthquake regions, and an authority in this field should be consulted when serious seismic conditions exist.

D. MEMBRANE DESIGN

7.9. Impervious Central Core.-A typical earth-core rockfill section using a central impervious earth core is shown on figure 7-9. Internal membranes of concrete, asphalt, and steel are not recommended because of the inability to inspect or repair them. The rockfill zones of the central core dam are discussed in sections 7.5, 7.6 and 7.7. The upstream rockfill material should be of sufficient size and quality to satisfy the riprap requirements discussed in section 6.22(c); however, riprap bedding requirements need not be met.

Earth-core rockfill dams are economical where site conditions suggest the use of rockfill, but preclude the use of a decked rockfill structure. This can be the case where the upstream abutments show highly weathered rock to great depths and thus present questionable cutoff conditions for an upstream membrane, or where the higher elevations of the abutments are covered with deep layers of overburden and preclude the economical installation of a positive trench-type cutoff for the decked rockfill.

The impervious material used in the core should be similar to the material used for earthfill cores, as discussed in chapter 6. The material should be placed at or near optimum moisture content in about g-inch lifts and should be compacted to 95 to 100 percent standard laboratory unit weight2 by a tamping roller. The plasticity index of the material should be sufficient to allow the core to deform without cracking. Specifications for this type of placement are given in appendix G.

Filter zones should be adequate to prevent piping of impervious material during steady-state or rapid drawdown conditions, and it is recommended that the filter criteria in section 6.10(i) be met. Multiple filters may be required if gradation differences between the core and rockfill materials are large. Figure 7-13 shows the placement of fine and coarse filter material for the 55-foot-high New Exchequer Saddle Dike in California.

The foundation and abutments against which the core rests should be carefully treated to prevent piping. Joints, cracks, fissures, and shear zones should be cleaned out to firm material and filled with concrete or grouted. Vertical faces, overhangs, and large rock protrusions should be flattened to slopes not steeper than 0.5:1, horizontal to vertical, by excavation or concrete placement [lo]. A trench type concrete cutoff wall may be necessary with central impervious earth cores when foundation grouting is required and the upper zone of rock is closely fractured, weathered, soft, etc.

The freeboard requirements are the same as those for earthfill dams discussed in section 6.21.

7.10. Reinforced Concrete.-More rockfill dams have been faced with conventionally placed reinforced concrete than with any other type of impervious membrane. In most cases, these facings have performed well for correctly compacted rockfill embankments; leakage has been within acceptable limits, and repairs have been minor. Slab thickness and reinforcing requirements have usually been determined by experience or precedent to satisfy the following criteria:

- Low permeability
- Sufficient strength to bridge subsided areas of the face
- High resistance to weathering
- Sufficient flexibility to tolerate small embankment settlement

Compacted rockfill dams have considerably reduced embankment settlement, and the use of a well-compacted facing layer, which acts as a continuous, firm bedding surface for the concrete face, has reduced the bridging requirements of the face.

For a small dam on a stable foundation, a reinforced concrete slab with a minimum thickness of 8 inches is recommended. The concrete should be dense, durable, weather resistant, and have low permeability (concrete mix designs are discussed in app. F). If foundation settlement could occur, or if other factors such as earthquake conditions exist, it would be wise to increase the membrane thickness.

The amount of steel reinforcing should meet the generally accepted requirements, 0.5 percent of the concrete area. The reinforcing should be placed both horizontally and vertically in a single layer in the center of the slab.

Because of the low reservoir head and the small amount of settlement expected, horizontal or vertical expansion joints are normally not required in the reinforced concrete facings for low dams. Vertical joints may be required to compensate for horizontal expansion on low dams of considerable length and are often used to facilitate construction of the face. Polyvinyl chloride or rubber water stops should be used to ensure impermeability along the joints.

The type of cutoff between the concrete facing and the foundation depends on the quality .of rock encountered. For sound rock, the doweled cutoff slab shown on figure 7-5 has demonstrated its adequacy and economy [11]; whereas, in closely jointed, weathered rock, or rock of questionable quality, a cutoff wall should be used. Water stops should be used between the cutoff and the facing.

Because concrete facings provide little resistance to wave run up, increased freeboard is required to prevent wave run up and over splash. Coping or parapet walls similar to that shown on figure 7 -10 may be used to reduce the height of embankment required for freeboard purposes. These walls should be constructed as integral continuations of the concrete face and reinforced accordingly. Coping walls work well. Cooke [11] reports that in one case, walls 10 feet high have stored water to 8 feet without harmful effects; although this procedure is not recommended for the types of structures considered here. When coping walls are used for run up and over splash protection, freeboard requirements of the embankment can be less than those for an earthfill dam (sec. 6.21). The design top of the rockfill should be above the maximum water surface.

The height of the coping wall can be determined either by precedent or the designer’s experience.

The spillway should be designed so that its capacity increases rapidly as the reservoir surface begins to encroach on the coping walls.

Concrete has generally been placed by the same slip-forming process used in road construction, but in some cases shotcrete has been used effectively.

Figure 7-14 shows the placement of concrete by the use of slip-forms on the upstream slope of New

Exchequer Dam in California, and figure 7-15 shows shotcreting at Taum Sauk Dam near St. Louis, Missouri.

Placement of the concrete membrane should not begin until the entire embankment has been placed; this allows maximum construction settlement and reduces the possibility of cracking and excess leakage. If concurrent slab placement is necessary to complete the job on time, an experienced dam designer should be consulted. Concrete overruns could occur as a result of the voids in the facing layer and should be accounted for in estimating the quantities.

7.11. Asphahic Concrete.-The second most common facing for rockfill dams is asphaltic concrete.

Asphaltic concrete provides more flexibility and can thus tolerate larger settlements than reinforced concrete facings. It offers an economical alternative to concrete and has proved dependable when correctly constructed. The recommended upstream slope for asphaltic-concrete-faced rockfill dams is 1.7:l or flatter, as shown on figure 7-10. The zone A material should provide a well-graded, free draining rock layer to eliminate uplift pressures in case of rapid drawdown. Yet it should also provide sufficient resistance to limit water velocities and prevent piping if a crack forms in the membrane.

The gradation of the zone A material should be smaller than the zone B material. A base course with a minimum thickness of 6 inches should be provided beneath the asphalt as a leveling course, working surface, and smooth base surface for asphalt placement. The base course should consist of well-graded material from a maximum size of about 1 to 2 inches to 5 to 7 percent passing the NO. 200 sieve. The base course should be well compacted by a vibratory roller. Figure 7-16 shows the completed rockfill section at Upper Blue River Dam before asphalt membrane placement.

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

The recommended asphaltic-concrete membrane thickness is between 4 and 12 inches, depending on the hydraulic head. It should be applied by a standard road paver in one to three approximately equal lifts, depending on the total thickness [19]. Figure 7-17 shows placement of an asphaltic-concrete layer at, Montgomery Dam in Colorado. A seal coat is desirable on the finished surface of the membrane.

The seal coat waterproofs the facing and provides increased durability. Each layer is placed in strips 10 to 12 feet wide and constructed at right angles to the axis of the dam. Paving is placed on the upslope pass only, and the machine is returned to the bottom and reloaded for each strip. If sufficient, asphaltic material is not available for each strip, reloading must be performed on the upstream face; a single paving machine should be capable of placing between 25 and 35 tons of asphaltic concrete per hour. Rolling operations should be performed shortly after placing. Smooth-wheel rollers, either the vibratory or standard tandem type, can be used for lift compaction.

Lifts should be compacted to a minimum of 97 percent of standard laboratory density. Construction control can be effected by taking core samples from the asphalt face at random locations and performing asphalt content, unit weight, stability, and permeability tests.

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

1. Apply a tack coat of asphaltic cement, the same type used in the mix.
2. Place the asphaltic concrete, overlapping the joint 3 to 6 inches.
3 Reheat the joint with an infrared heater, avoiding open flames.
4. Compact the joint by rolling, immediately after reheating.

When one lift is placed on top of another, the parallel joints in the strips of the top lift should be offset 3 or 4 feet from the joints of the bottom strip.

The foundation cutoff used with asphalt facings must promote easy placement of the asphalt lift at the contact with the foundation. A trench-type cutoff wall similar to that shown on figure 7 -3 is recommended.

The cutoff used at Montgomery Dam is shown on figure 7-18; the 12-inch-diameter drain was used to reduce uplift pressure during drawdown.

The cutoff used at Upper Blue River Dam is visible at the left edge of figure 7-16.

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

- Durable

- Flexible
- Impervious
- Does not creep
- Resists weathering

Material found within an economical hauling distance of the dam should be used in the asphaltic concrete if possible. A number of different materials and gradations ranging from silty sand [20] to graded gravel [17] have been used to construct adequate upstream facings. Clay fines should not be permitted in mixes because the clay tends to ball during the drying process and to crush when compacted, thereby leaving dry material exposed to the reservoir water.

The gradation limits of the material used for the asphaltic-concrete facing at Montgomery Dam are shown in table 7 -1.

Tests should be performed using various gradations, compressive efforts, percent asphaltic cement, and percent lime to determine the mix that has the maximum unit weight and best satisfies the above criteria. In certain cases, 1 to 3 percent lime has been shown to reduce the underwater expansion of the asphaltic concrete, thereby increasing its life expectancy.

The tests that may be performed to evaluate the materials and the different mixes are:

1. Sieve analysis and specific gravity test
2. Immersion-compression test
3. Unconfined-compression test
4. Sustained-load test
5. Permeability test
6. Wave-action test

Tests 1 through 5 are similar to those described in ASTM Standards, part II. Test 6, concerning the effect of wave action, is described in [20, 21, 22]; this test was developed by the Bureau of Reclamation

to simulate wave effects on asphaltic concrete facings and has proved useful in helping to select the correct mix proportions.

Special tests that can be performed are:

1. Slope-flow test
2. Coefficient of expansion test
3. Flexural strength test

4. Effect of reservoir ice test

All of the above tests, both standard and special, were performed by the Bureau of Reclamation to determine the proper type of asphaltic cement and the correct percentage to be used at Montgomery Dam [21]. These tests led to a mix design incorporating 8.5 percent asphaltic cement. The specifications for the asphaltic cement used at Montgomery Dam are reprinted below:

"All asphalt for use in the asphaltic concrete shall be uniform in character, shall not foam when heated to 350 ^{0}F, and shall conform to the following specifications and requirements;

(a) Penetration (tested per ASTM D 5)

(1) At 77 ^{0}F, 100 grams, 5 seconds 50-60

(2) At 32 ^{0}F, 200 grams, 60 seconds min. 12

(b) Ductility at 77 ^{0}F, 5 cm/min min.140 cm (tested per ASTM D 113)

(c) Flash point (Cleveland open cup) min. 450 ^{0}F (tested per ASTM D 92)

(d) Solubility in carbon tetrachloride min. 99.5 % (tested per ASTM D 165)

(e) Softening point (ring and ball method) min. 125 ^{0}F (tested per ASTM D 36)

(f) Spot test (per paragraph 3 of AASHTO Specifications T 102) negative

(g) Results of tests made on residues after thin film oven heating, per test method No. Calif. 337-A, January 3, 1956, of the Division of Highways, Department of Public Works, State of California, shall conform to the following as compared with like tests made on the identical material before such heating.

Such method shall be considered to be a part here of, provided that in lieu of the pertinent AASHTO Specifications there shall be used the ASTM Specifications referred to in this item plus ASTM D 6 and E 11. The contractor shall cause the producer of the asphalt to supply the engineer, as required, with data or curves showing the relation between temperature and viscosity representative of the asphalt as furnished for the work."

The results of the tests specified above are shown in table 7-2.

A very low air-void content resulting from proper mix design and compaction is required to obtain durable facings; however, a low air-void ratio cannot be obtained by simply adding more asphalt cement.

Air-void ratios of 1 percent are commonly obtained, and the maximum air-void ratio allowed in the construction of an asphalt facing should be 5 percent.

Experience has indicated that densely graded aggregates with ample filler (minus No. 200), correctly proportioned with a 50 to 60 penetration, paving grade asphalt cement produces a very workable, relatively easily compacted hot mix at about 300 “F.

The thicker films of asphalt cement obtained with a rich mix of slightly harder 50 to 60 asphalt cement, as compared with an 85 to 100 or 100 to 150 penetration asphalt cement, can be expected to increase water tightness, stability, and durability. The completed asphaltic-concrete facing at Upper Blue River Dam is shown on figure 7-19.

Parapet walls should be used with asphaltic-concrete facings in lieu of increasing the height of the dam to retard wave run up and over splash. Galvanized corrugated metal has been used for a number of small dams [17,211 and appears to be performing well; figure 7-19 shows the parapet wall at Upper Blue River Dam. When parapet walls are used to protect against wave run up and over splash, the freeboard heights of the embankment may be reduced from those heights required for earthfill dams (sec. 6.21); however, the embankment crest must be above maximum water surface. Wall heights can be determined by precedent or design experience.

For further information on asphalt facings, the reader should consult the references at the end of this chapter. Specifications for materials used to manufacture asphaltic concrete are subject to change, and the literature should be consulted.

7.12. Steel.-Steel facings have been used on relatively few dams throughout the world, but their satisfactory performance on these few dams illustrates clearly that they should be given serious economic consideration by dam designers. Few design criteria besides precedent are applicable, and the available literature should be consulted for a complete review of the practices used [18, 23, 24, 251.

Figure 7-20 shows the upstream face of the Bureau of Reclamation’s El Vado Dam where the steel plate is in excellent condition after 45 years of service.

Steel-faced dams can be rapidly constructed and should be capable of tolerating greater embankment movements than either concrete or asphalt-faced dams. The most prominent disadvantage to steel facings is the possibility of corrosion reducing their economic life, although this can be effectively controlled by cathodic protection on both faces of the plate. Experience with the few existing steel-faced dams strongly indicates that corrosion failure of the plate is remote and that for practical purposes, the facing can be assumed to be permanent if proper maintenance is provided.

Steel-faced dams have generally been constructed with upstream slopes from 1.3: l to 1.7:1.

For rockfill dams, the upstream and downstream slopes need not be flatter than the natural slope of the material, which is generally from 1.3:1 to 1.4:1.

The steeper slopes lead to reduced costs, but slightly increased face construction difficulties.

The portion of the embankment on which the steel plate bears (zone A on fig. 7-10) should, in general, be constructed of well-graded, pervious gravel to provide a uniform bearing surface for the steel facing. Anchor rods should extend from the facing plates into the embankment to prevent uplift or loosening of the face as a result of embankment settlement or wave action. 1\vo methods of anchoring the faceplate to the embankment have been used. The first method requires that the steel plate be constructed on a scaffolding grid that is raised a few feet off the face of the dam; bedding material is then placed between the facing and the embankment either after the plate construction is completed or concurrently with its construction up the face. The second method requires that anchor bar holes be dug in the completed faceplate bedding material, and the anchor bars grouted in with concrete.

The first method provides more adequate plate anchoring, but the second method is less costly.

The steel plates should have a thickness of 14 to 3/8 inch, depending on the assumed magnitude of the movements of the dam. All joints and seams should have a continuous fillet weld, and nuts used on alignment or anchor bolts should be welded to the plate on all sides to prevent leakage. Plate sizes have varied considerably; large plates are more difficult to handle and appear to provide little advantage over smaller plates.

The foundation cutoff used should be the trench type cutoff wall shown on figure 7 -4. It should be designed to withstand the tension caused by settlement and stresses imposed by any differential face movement adjacent to the cutoff. Coping walls should be used to retard over splash. Freeboard requirements are similar to those for a concrete-faced rockfill dam (sec. 7.10).

Expansion joints are usually V -shaped metal strips placed perpendicular to the axis of the dam and extending from the crest to the cutoff; the V strips may have their raised portion placed on either the front or back of the steel face.

Module IV

Diversion Head works: Components, Weir, Design of impervious floor, Khosla's theory Canal Regulations works: Canal Fall, its type and design methods, Canal outlets.

Diversion works

9.1 Weirs and barrages

9.1.1 General

Weirs and barrages are relatively low-level dams constructed across a river to raise the river level sufficiently or to divert the flow in full, or in part, into a supply canal or conduit for the purposes of irrigation, power generation, navigation, flood control, domestic and industrial uses, etc. These diversion structures usually provide a small storage capacity. In general, weirs (with or without gates) are bulkier than barrages, whereas barrages are always gate controlled. Barrages generally include canal regulators, low-level sluices to maintain a proper approach flow to the regulators, silt excluder tunnels to control silt entry into the canal and fish ladders for migratory fish movements.

Weirs are also used to divert flash floods to the irrigated areas or for ground water recharging purposes. They are also sometimes used as flow measuring structures. Figure 9.10 gives a detailed description of the various parts of a typical barrage constructed on rivers flowing over permeable beds (see also Baban, 1995).

The site selection of a barrage depends mainly on the location and elevation of the off-take canal, and a site must be selected where the river bed is comparatively narrow and relatively stable. The pondage requirement and interference with the existing structures such as bridges, urban development, valuable farmland, etc., must be considered, as well as available options to divert the flow during construction. Cofferdams are temporary structures used to divert water from an area where a permanent structure has to be constructed. They must be as watertight as practicable, relatively cheap and, if possible, constructed of locally available materials.

Diversion facilities such as tunnels or canals, provided to divert the flow from the site area, are sometimes used as part of the permanent facilities (e.g. penstocks, spillways, sluices, conveyances to turbines, or discharge channels from turbines, etc.). If the construction work proceeds in two stages, part of the structure completed in the first stage may be used as a diversion facility (spillway or sluice) during the second stage of construction (Fig. 9.1) (Linsley and Franzini, 1979; Vischer and Hager, 1997).

The selection of the design flood for these diversion works depends on the risk that one is prepared to take (see equation (4.1)). For example, a more conservative design flood has to be considered for situations where overtopping during construction would have disastrous results. If overtopping is permitted, care must be taken to strengthen the top and the downstream slope of the cofferdam to minimize erosion. The overtopping flow must be spread over the longest possible length of the cofferdam, thus reducing the concentration of flow. The control of floating debris is another essential requirement in order to minimize the clogging of diversion tunnels, especially during the high-flow season.

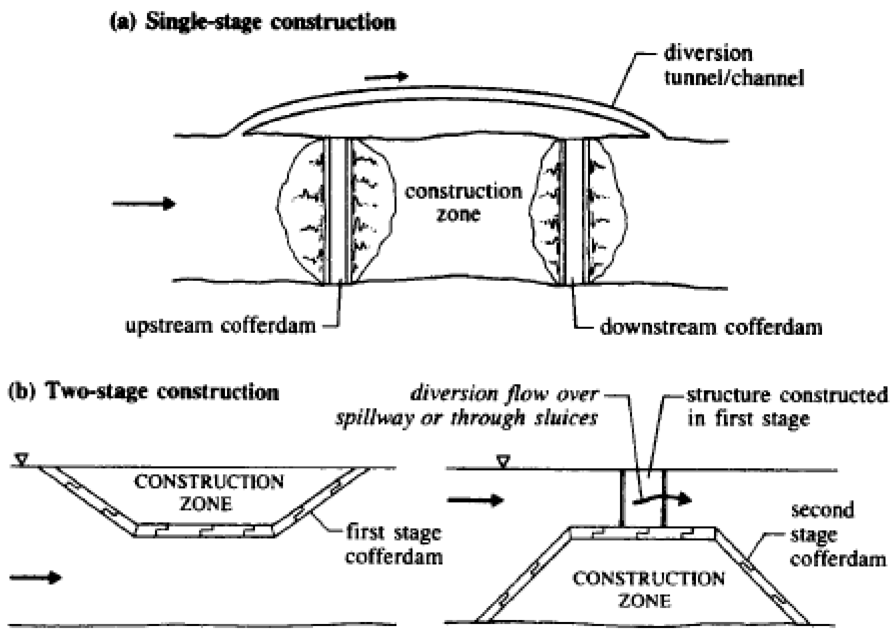


Fig. 9.1 Streamflow diversion by cofferdams

9.1.3 Barrage components

(a) Guide banks

Guide banks direct the main river flow as centrally as possible to the diversion structure. They also safeguard the barrage from erosion and may be designed so that a desirable curvature is induced to the flow for silt exclusion from the canals. The side slopes of the guide banks must be protected by stone pitching, with a sufficient ‘self-launching’ stone apron at the lowest feasible level (Fig. 9.2). The top levels of the guide banks will depend on the maximum increase in the flood level upstream of the barrage. The afflux (level difference between the headwater and tail water during the passage of maximum flood flow) results in a backwater curve upstream of the barrage, and flood banks have to be provided along the upstream reach of the river to contain the flood flow.

(b) Wing walls

Wing walls flanking the barrage and supporting the abutting earth bunds are designed as retaining walls. Cut-off walls (taken below the scour levels) below the wings and abutment walls at both sides, in addition to the upstream and downstream sheet pile cut-offs across the river, form an enclosed compartment providing good weir foundation conditions.

(c) Gates

Gates used on barrages are of the same type as those used on spillway crests (Chapter 6). Vertical lift and Tainter gates are most frequently used to control the flow rate over the crest (sill) of a barrage. The discharge capacity of a gated crest depends on the conditions of free (Chapter 4) orsubmerged flows below the gate. For completely raised gates, Table 8.1 for free flow and Table 9.1 for submerged flow conditions should be referred to. Section 6.5 and equations (6.2)–(6.5) together with Figs 6.9 and 6.10 summarize the hydraulic conditions for flow under and over vertical lift and Tainter gates.

(d) Regulators

The structures controlling diversion into a supply canal are called regulators (Section 9.2). The design principles are the same as those used in the design of barrages, except that the regulators are a smaller version of barrages. The entry sill of a regulator must be such that it permits entry of the maximum flow at various pondage levels. Another important consideration in designing the regulator is silt exclusion from canals (Section 9.2). Silt-excluder tunnels are often provided in the barrage bays adjacent to the regulator, so that the heavier silt-laden bottom layers of water bypass through the tunnels (Fig. 9.3).

(e) Dividing wall

The dividing wall is built at right angles to the axis of the weir, separating the weir and the under-sluices (Fig. 9.3). It usually extends upstream beyond the beginning of the regulator and downstream to the launching apron (talus). The sluice bay floor level is generally kept as low as possible to create pool conditions (for silt settlement and its exclusion) and the dividing wall separates the two floor levels of the weir. The downstream extension of the dividing wall provides a barrier between the stilling basin and scouring bay, in order to avoid cross-currents. A properly designed dividing wall can also induce desirable curvature to the flow for sediment exclusion from the canal-head regulator. The dividing wall may also serve as one of the side walls of the fish ladders (Section 9.3) and be used as a log chute.

(f) Weir block and stilling basin

The weir block of the barrage is designed either as a gravity structure (the entire uplift pressure due to seepage is resisted by the weight of the floor) or as a non-gravity structure (the floor, relatively thinner, resists the uplift by bending). It may be of different forms, e.g. a sloping weir with upstream and downstream glacis, a vertical drop weir, an ogee weir, or a labyrinth weir (zig-zag crest). In some cases the barrage has a raised sill (crest) (e.g. Fig. 9.10). The advantage of this arrangement is a reduced height of the gates; however, the height of the sill must not exceed a value which could result in a raised (maximum) upstream water level. A hydraulically suitable design for a low sill is the so called 'Jambor sill' (Jambor 1959) which consists of a part-cylindrical shape with a smooth upstream and downstream transition (typically a sloping apron with a slope 1 : 2 to 1 : 2.25). For a height s of the crest above the approach channel bed (e.g. 3.69 in Fig. 9.10) and the upstream (approach) depth y_0 (6.00 in Fig. 9.10) computation using the potential flow theory will result in a radius R of the cylindrical sill which would not influence (reduce) the discharge capacity of the structure (compared with the case without the raised sill) for a given upstream water level. Typical values for s/y_0 are $0.15 < s/y_0 < 0.3$ resulting in $2.5 < R/s < 15$. For further details of the computation, particularly for the case of non-modular flow see e.g. Dolezˇal (1968). The sloping surface downstream of the weir crest (glacis) and its transition into the stilling basin should be designed so that a hydraulic jump occurs on it over the full range of discharges. Details of stilling basin design are given in Chapter 5. Protection works, such as cut-off piles, aprons and an inverted filter, are provided both upstream and downstream of the impervious floor of the weir block (see also worked examples 9.1 and 9.2). Chapters 5 and 8 also provide further details of the design of low-head weirs.

(g) Navigation lock

Special provisions must be made at the barrage site if the river is navigable. Navigation locks with appropriate approaches, etc. must be provided (Chapter 11).

9.1.4 Failures of weir foundations on permeable soils and their remedies (see also Section 2.6)

(a) Exit gradient (G_e) and piping

The exit gradient is the hydraulic gradient (Fig. 9.4) of the seepage flow under the base of the weir floor. The rate of seepage increases with the increase in exit gradient, and such an increase would cause ‘boiling’ of surface soil, the soil being washed away by the percolating water. The flow concentrates into the resulting depression thus removing more soil and creating progressive scour backwards (i.e. upstream). This phenomenon is called ‘piping’, and eventually undermines the weir foundations.

The exit gradient (tan_; Fig. 9.4) according to the creep flow theory proposed by Bligh (Khosla, Bose and Taylor, 1954) is

$$G_e = H_s/L \qquad (9.1)$$

where L is the total creep length equal to $2d1<b<2d2$, $d1$ and $d2$ being the depths of the upstream and downstream cut-off piles respectively and b the horizontal floor length between the two piles; Hs, the seepage head, is the difference in the water levels upstream and downstream of the weir.

The piping phenomenon can be minimized by reducing the exit gradient, i.e. by increasing the creep length. The creep length can be increased by increasing the impervious floor length and by providing upstream and downstream cut-off piles (Fig. 9.4).

(b) Uplift pressures

The base of the impervious floor is subjected to uplift pressures as the water seeps through below it. The uplift upstream of the weir is balanced by the weight of water standing above the floor in the pond (Fig. 9.5), whereas on the downstream side there may not be any such balancing water weight. The design consideration must assume the worst possible loading conditions, i.e. when the gates are closed and the downstream side is practically dry.

The impervious base floor may crack or rupture if its weight is not sufficient to resist the uplift pressure. Any rupture thus developed in turn reduces the effective length of the impervious floor (i.e. reduction in creep length), which increases the exit gradient.

The provision of increased creep lengths and sufficient floor thickness prevents this kind of failure. Excessively thick foundations are costly to construct below the river bed under water. Hence, piers can sometimes be extended up to the end of the downstream apron and thin reinforced concrete floors provided between the piers to resist failure by bending.

The two criteria for the design of the impervious floor are as follows.

1. *Safety against piping.* The creep length is given by

$$L = cH_s \qquad (9.2)$$

where c is the coefficient of creep (_1/*G*e).

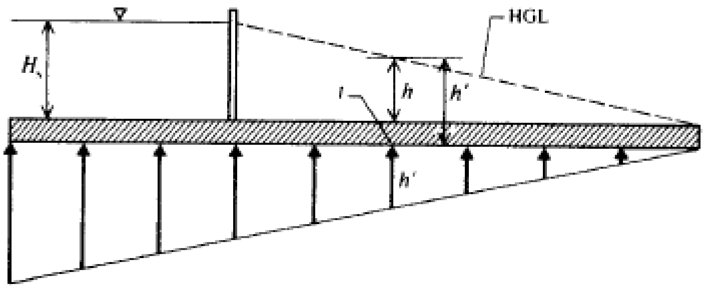


Fig. 9.5 Uplift pressure under impervious floor

2. *Safety against uplift pressure (Fig. 9.5).* If *h*_ is the uplift pressure head at a point under the floor, the pressure intensity is

$$p = \rho g h' \ (\mathrm{N\,m^{-2}}). \qquad (9.3)$$

This is to be resisted by the weight of the floor, the thickness of which is *t* and the density _m (for concrete, ρ_m=2240kgm/m^3). Therefore,

$$\rho_m g t = \rho g h'$$

giving

$$h' = S_m t$$

where *S*m is the relative density of the floor material. Thus we can write

$$h' - t = S_m t - t$$

which gives

$$t = (h' - t)/(S_m - 1) = h/(S_m - 1) \qquad (9.4)$$

where *h* is the pressure head (ordinate of hydraulic gradient) measured above the top of the floor. A safety factor of around 1.5 is usually adopted, thus giving the design thickness of the concrete floor as

$$t \simeq 1.2h. \qquad (9.5)$$

The design will be economical if the greater part of the creep length (i.e. of the impervious floor) is provided upstream of the weir where nominal floor thickness would be sufficient.

The stilling basin area of the weir is subjected to low pressures (owing to high velocities) which, when combined with excessive uplift pressures, may rupture the floor if it is of insufficient thickness. Usually, the floor is constructed in mass concrete without any joints, and with a hard top surface to resist the scouring velocities over it.

(c) Approach slab

The provision of a concrete slab upstream of the weir section (sloping down gradually from the 2 horizontal to 1 vertical slope of the crest section – Fig. 9.10) increases the seepage length, with a corresponding reduction in the exit gradient. It will also provide a smooth erosion resistant transition for the accelerating flows approaching the weir. The upstream end of the approach slab is firmly secured to the upstream sheet piling or to the vertical concrete cut-off. It is usually monolithic with the weir section in order to provide additional resistance to sliding.

9.1.5 Pressure distribution under the foundation floor of a weir/barrage

Applying the theory of complex variables (of potential theory) involved with the seepage flow under a flat floor (see Section 2.6) a Laplace differential equation can be formulated which on integration with appropriate boundary conditions suggests that the pressure head (*P*) at any point beneath the floor is a fraction, ϕ, of the total head, *H*s (see Fig. 9.4). Thus a solution to Laplace equation (Khosla *et al.*, 1954; Leliavsky, 1965) can be written as

$$\phi = P/H_s = (1/\pi)\cos^{-1}(2x/b) \tag{9.6}$$

for the underside of the floor where *b* is the total floor length and *x* is the distance from the centre of the floor to the point where the uplift pressure head is *P*.

Equation (9.6), based on the potential theory of the seepage flow, suggests entirely different distribution to that of Bligh's creep theory (i.e. linear distribution – see Fig. 9.5).

In reality the weir foundations are composite in construction consisting of floor slabs (horizontal or sloping), pilings or cut-off structures and a direct solution of the Laplace equation is not feasible. Khosla *et al.* (1954), in dealing with this problem, introduced a method of independent variables splitting the composite weir/barrage section into a number of simple forms of known analytical solutions and by applying some corrections in transferring the results to the composite section.

The simple standard forms of a composite section are:

i) a straight horizontal floor of negligible thickness with a sheet pile at either end;

ii) a straight horizontal floor of finite thickness (depressed floor) with no cut-off piles;

iii) a straight horizontal floor of negligible thickness with an intermediate sheet pile.

Khosla *et al.* produced pressure charts (see Figs 9.6 and 9.7) from which the pressures at key points (junction points of the floor and the pile and the bottom corners of the depressed floor) of these elementary forms can be read off. These are then corrected for the mutual interference of piles, the floor thickness and the slope of the floor, if any.

It must be realized that this procedure assumes throughout a uniform permeability coefficient (k) (see Section 2.3.4).

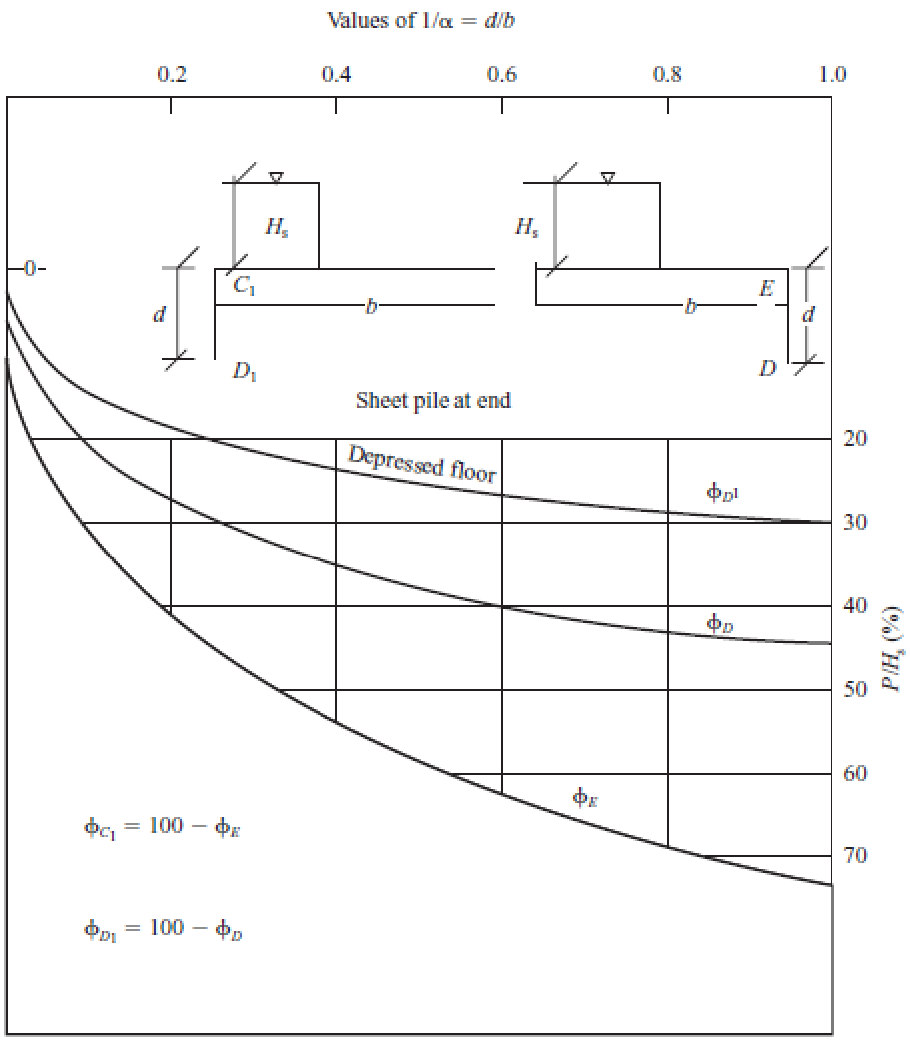


Fig. 9.6 Khosla's pressure chart with end pile

(a) Pressure distribution corrections

(i) CORRECTION FOR MUTUAL INTERFERENCE OF PILES (C%) (SEE FIG. 9.8)

$$C = 19\sqrt{D/b_1}(d + D)/b \qquad (9.7)$$

where *D*=the depth of the pile line whose influence has to be determined on the neighboring pile of depth *d*, *d*=the depth of the pile on which the effect of the pile depth *D* is required to be found, *b*=the total length of the floor, and *b*1=the distance between the two mutually interfering piles.

The correction computed by equation (9.7) is additive or subtractive depending upon whether the pile *d* is located upstream or downstream of pile *D*.

In the case of the intermediate piling shallower than the end pile ($D<d$) and $b1>=2d$ the mutual interference is negligible.

(ii) CORRECTION FOR FLOOR THICKNESS

The correction for floor thickness is interpolated linearly over the length of the piling once the pressures at the key points are computed using Figs 9.6 and 9.7. In equation (9.7) when computing the *corrections to the mutual interference with floor thickness* only the net depths of the pilings (i.e. *D*-*t*, *d*-*t*, where *t* is the floor thickness) instead of *D* and *d* are used.

(iii) CORRECTION FOR THE SLOPING FLOOR

The depths of the pilings to be used in the correction equation (equation

(9.7)) are always measured from the top level of the piling, for which the correction is computed – see worked example 9.2.

The slope correction is only applicable to the key points of the pile line fixed at the beginning or the end of the slope. Correction charts following this approach were constructed by Khosla *et al.* (1954).

The (theoretical) percentage correction for sloping floors is negligible for slopes less than 1:7; it is 2.3<(%)<4.5 for slopes 1:7 to 1:3, 6.5% for slope 1:2 and 11.2% for 1:1; the actual correction is obtained by multiplying the appropriate value by *b*2/*b*1, *b*2 being the horizontal distance of the sloping floor. This correction is additive for down slopes (following the direction of seepage flow) and subtractive for rising slopes.

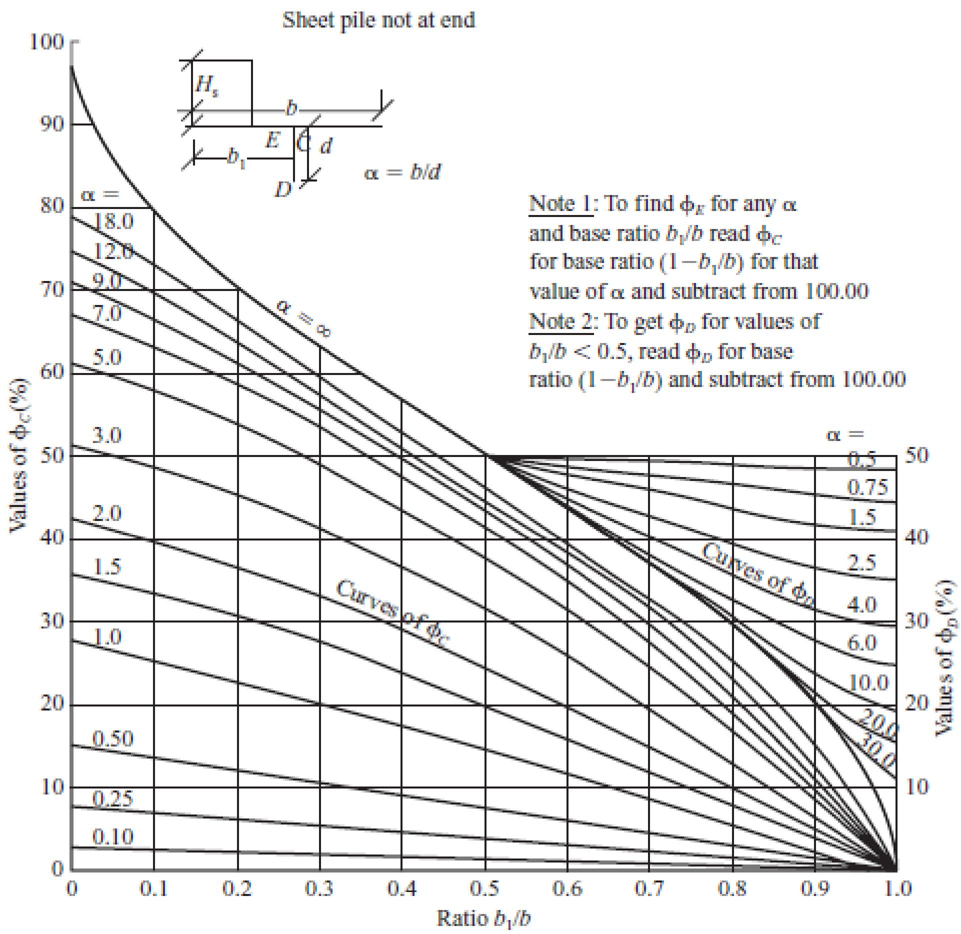


Fig. 9.7 Khosla's pressure chart with intermediate pile

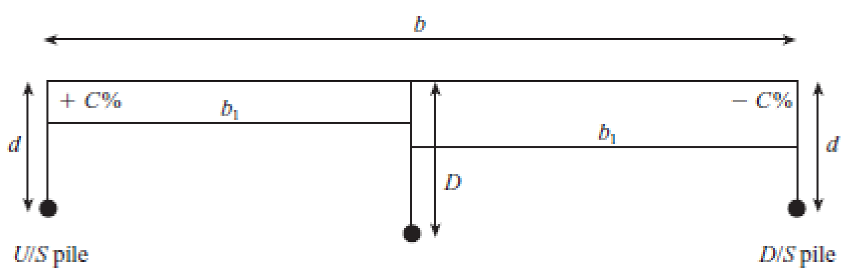


Fig. 9.8 Mutual interference of piles

(b) Actual exit gradient (Ge)

On the basis of the potential theory of flow Khosla *et al.* (1954) suggested the actual exit gradient given by (with one downstream end cut-off pile of depth d with the total base length of b)

$$G_e = H_s/\pi d\sqrt{\lambda} \quad (9.8)$$

where $\lambda(1/2(1+\alpha^2)^{1/2})$, α being b/d.

For alluvial soils the critical (safe) exit gradient is around 1 in 1, and with a safety factor of around 5.5 the permissible exit gradients are about1:6 to 1:7 for fine sand, 1:5 to 1:6 for coarse sand and 1:4 to 1:5 for shingle.

9.1.6 Barrage width and scour depth

(a) Barrage width

The barrage width must be sufficient to pass the design flood safely. The present trend is to design the barrage for a 100–150 year frequency flood and provide a breaching section along the main earth bund, located at a safe distance from the barrage itself. The breach section acts as a fuse plug, and its provision is a more economical solution than providing a large spillway capacity in the barrage.

The minimum stable width of an alluvial channel is given by the regime equation (Chapter 8)

$$B = 4.75Q^{1/2} \quad (9.9)$$

where B is the waterway width (measured along the water surface and at right angles to the banks) in metres, and Q is the maximum flood discharge in m^3/s.

(b) Regime scour depths

The river bed is scoured during flood flows and large scour holes (not to be confused with local scour, Section 5.3.3) may develop progressively adjacent to the concrete aprons which may cause undermining of the weir structure. Such flood scour depth below HFL corresponding to a regime width (equation (9.9)) is called regime scour depth (or more precisely regime hydraulic radius), *R*s, estimated by the following (Lacey's) formula

$$R_s = 0.475(Q/f)^{1/3} \quad (9.10)$$

if the actual waterway provided is greater or equal to the regime width (equation (9.9)) and

$$R_s = 1.35(q^2/f)^{1/3} \quad (9.11)$$

if the waterway provided is less than the regime width, where *R*s is measured from the high flood level (HFL) and f is Laceys silt factor (Singh, 1975):

$$f = 1.75d^{1/2} \quad (9.12)$$

where d is the mean diameter of the bed material (in mm) and q is the discharge per unit width of channel (see also Section 8.3).

Weir failure due to scour can be prevented by extending the sheet pile cut-offs to a level sufficiently below the regime scour depth across the full width of the river (Fig. 9.9).

(c) Concrete aprons and inverted filter

The aprons are of plain concrete blocks of about 1m×1.5m×0.75m deep, cast *in situ*. The downstream apron is laid with 70–100 mm open joints filled with spawls (broken stones), so that the uplift pressure is relieved. An inverted filter of well-graded gravel and sand is placed under the concrete apron (Fig. 9.9) in order to prevent the loss of soil through the joints. The upstream apron is laid watertight so that the uplift pressure and downward flow is reduced (due to the increase in creep length). Aprons of boulder or stone are laid downstream and upstream of the concrete aprons (Fig. 9.10).

9.1.7 Effect of barrages on river water quality

The flow of water over and/or under barrage gates as well as over spillways and weirs results in aeration with a beneficial effect on oxygen concentration levels in the downstream river reach.

The most frequent case is aeration at overfalls. On the basis of laboratory experiments (Avery and Novak, 1978) with results corroborated by extensive field measurements (Novak and Gabriel, 1997) the oxygen deficit ratio r (ratio of upstream to downstream oxygen deficit) at 15 °C is given by

$$r_{15} - 1 = kFr_J^{1.78}Re_J^{0.53} \qquad (9.13)$$

where k=0.627×10_4 (for water without salinity), $Fr_J = (gh^3/2q_J^2)^{0.25}$ and $Re_J = q_J/\nu$, h is the difference between the upstream and downstream water level and qJ is the specific discharge (m^2 /s) at impact into the downstream pool; this equals the specific discharge at a solid weir crest (q=qJ) but with air access below the nappe (e.g. at flow over gates) q=2qJ. Equation (9.13) (which is dimensionless) can be transformed for the given value of (k) (h (m), q (m^2 /s)) into:

$$r - 1 = 0.18\, h^{1.34}\, q^{-0.36} \qquad (9.13a)$$

The limits of applicability of equation (9.13) are h_6q1/3 (m) (equation (5.6) should be referred to, i.e. the overfalling jet should not disintegrate) and the downstream pool depth d should be bigger than 0.0041 ReJ0.39FrJ0.24 (m) (or d>0.909 h0.18 q0.27 (m, m^2/s)) (as the major part of oxygen uptake occurs in the downstream pool).

For the outflow under a gate with a hydraulic jump the deficit ratio is

$$r_{15} - 1 = Fr_1^{2.1} Re^{0.75} \tag{9.14}$$

($Fr1$ is the jump supercritical Froude number and $Re=q/v$).

The temperature correction from r_T to r_{15} is given by

$$(r_T - 1)/(r_{15} - 1) = (1 + 0.046T)/1.69. \tag{9.15}$$

Gulliver *et al.* (1998) reviewed various prediction equations for oxygen transfer at hydraulic structures and concluded that for flow over sharp crested weirs equation (9.13) in the form (Fr=8gh3/q2)0.25, $Re=q/v$)

$$E_{20} = 1 - (1/(1 + 0.24 \times 10^{-4} Fr^{1.78} Re^{0.53}))^{1.115} \tag{9.16}$$

gives the best results when tested against field measurements ($E20$ is the transfer efficiency indexed at 20 °C E=1-(1/π).

For ogee spillway crests they recommend

$$E_{20} = 1 - \exp(-0.263h/(1 + 0.215q) - 0.203d) \tag{9.17}$$

and for gated sills

$$E_{20} = -1 - \exp[-0.0086(hq/s) - 0.118] \tag{9.18}$$

where s is the submergence of the gate lip.

For oxygen transfer at cascades see Chanson (1994) and for further treatment of the whole subject see Novak (1994).

Aeration at hydraulic structures is usually, but not always, beneficial. If the upstream water is fully or nearly saturated with oxygen then further oxygen enrichment can lead to oxygen super-saturation that may have detrimental effects as it can cause gas bubble disease in fish. This situation is more likely to occur at high head structures with high velocities of flow than at barrages. The problem can be alleviated by some structural measures and in any case is mostly very localized and does not propagate far downstream of the dam.

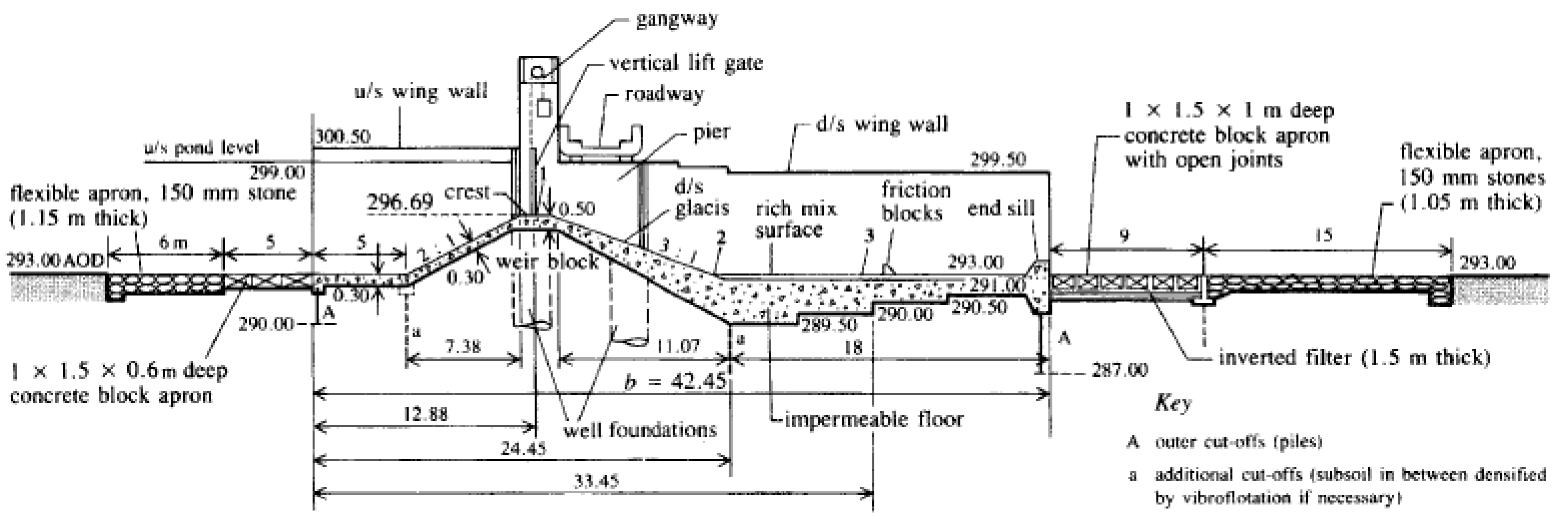


Fig. 9.10 Cross-section through a barrage showing details of the foundations; all dimensions in metres

9.2 Intakes

9.2.1 Introduction

The intake structure (or head regulator) is a hydraulic device constructed at the head of an irrigation or power canal, or a tunnel conduit through which the flow is diverted from the original source such as a reservoir or a river. The main purposes of the intake structure are (a) to admit and regulate water from the source, and possibly to meter the flow rate, (b) to minimize the silting of the canal, i.e. to control the sediment entry into the canal at its intake, and (c) to prevent the clogging of the entrance with floating debris.

In high-head structures the intake can be either an integral part of a dam or separate; for example, in the form of a tower with entry ports at various levels which may aid flow regulation when there is a wide range of fluctuations of reservoir water level. Such a provision of multilevel entry also permits the withdrawal of water of a desired quality.

The layout of a typical intake structure on a river carrying a heavy bed load is shown in Fig. 9.16. The following are its major appurtenances:

1. the raised inlet sill to prevent entry of the bed load of the river;
2. the skimmer wall (with splitter pier) at the inlet to trap floating ice and debris;
3. the coarse rack (trash rack) to trap subsurface trash, equipped with either manual or automatic power-driven rack cleaning devices;
4. the settling basin (sand trap) followed by a secondary sill (entrance sill) diverting the bottom (sediment-laden) layers towards the desilting canal;
5. the flushing (desilting) sluice to flush the deposited silt;
6. the intake (head regulator) gates to control the flow rate into the canal;
7. the scouring (tunnel) sluices in the diversion weir to flush the bed load upstream of the inlet sill.

The desilting canal with its flushing sluices may be omitted if the sediment load which would settle in the settling basin is negligible; however, smaller grain size sediment (silt) is always likely to enter the canal, and maintenance of minimum velocities in the canal is essential to avoid its silting up.

9.2.2 Location and alignment of an intake

The river reach upstream of the intake should be well established with stable banks. As the bottom layers of the flow around a bend are swept towards its inside (convex) bank (see Section

8.3), it is obvious that the best location for an intake (to avoid bed load entry) is the outer (concave) bank, with the intake located towards the downstream end of the bend.

This choice of location from the sediment exclusion point of view is not always possible, and other considerations such as the pond (command) levels and their variations, navigation hazards, and location of the diversion structure, pump/power house, and outfalls must be considered.

An offtake at 90° to the main flow is the least desirable one. The structure should be aligned to produce a suitable curvature of flow into the intake, and a diversion angle of around 30°–45° is usually recommended to produce this effect; in addition, an artificial bend (Fig. 9.17), a groyne island (Fig. 9.18) or guide vanes (Fig. 9.19) may be designed to cause the required curvature of flow (see Avery, 1989). Model tests are desirable in deciding on the location and alignment of any major intake structure (Novak and Cˇ abelka, 1981).

The entrance losses at an intake depend upon the change in direction of the flow (entering the intake), the extent of contraction and the type of trash rack provided at the inlet. They are expressed in terms of the velocity head as *KV*2/2*g*.

The entrance loss due to a change in direction of flow (intake at an angle α with the main stream) is given by

$$\Delta h_\alpha = V^2/2g - \epsilon V_0^2/2g \qquad (9.19)$$

where *V*0 is the velocity of the main stream at the inlet, and is around 0.4 for α=90° and 0.8 for α=30°.

In the case of the inlet having a sill constructed with curved abutments and piers, the head loss, Δ*h*c, is given by

$$\Delta h_c \simeq 0.3V^2/2g. \qquad (9.20)$$

Equations (9.19) and (9.20) suggest the maximum entrance loss at the inlet:

$$\Delta h_e = 1.3V^2/2g - \epsilon V_0^2/2g. \qquad (9.21)$$

The rack losses,Δ*h*r, can be expressed by (Fig. 9.20)

$$\Delta h_r = \beta(s/b)^{4/3} \sin \delta V^2/2g \text{ (Kirschmer's formula)} \qquad (9.22)$$

(with flow parallel to rack bars), where βis a coefficient which depends on the type of rack bar (Table 9.2).

Meusburger (2002) provides a comprehensive discussion of inlet rack losses including an extensive bibliography.

9.2.3 Silt control at headworks

(a) Silt excluder

The silt excluder is a device constructed in the river bed just upstream of the regulator to exclude silt from the water (source) entering the canal. It is so designed that the top and bottom layers of flow are separated with the least possible disturbance, the top sediment-free water being led towards the canal while the bottom sediment-laden water is discharged downstream of the diversion structure through under-sluices. The device basically consists of a number of tunnels (Fig. 9.21) in the floor of the deep pocket of the river, isolated by a dividing wall. The sill level of the regulator is kept the same as that of the top level of the roof slab of the tunnels.

The capacity of the tunnel(s) is usually kept at about 20% of the canal discharge, and they are designed to maintain a minimum velocity of 2–3m/s(to avoid deposition in tunnels).

(b) Silt ejector or extractor

The silt ejector is a device constructed on the canal downstream of the head regulator but upstream of the settling basin (if any), by which the silt, after it has entered the canal, is extracted.

1. *Vane type ejector.* The layout of a vane type ejector is shown in Fig. 9.22. A diaphragm at the canal bed separates the top layers from the bottom ones. On entering the depressed area of the canal bed, the bottom sediment-laden layers are diverted by the curved vanes towards the escape chamber. The design should be such that the entry disturbances are minimal; the streamlined vane passages accelerate the flow through them, thus avoiding deposition.

2. *Vortex tube type ejector.* The vortex tube ejector (Fig. 9.23) consists of a pipe with a slit along its top, placed across the bottom of the canal at an angle of around 30°–90° to the direction of flow.

The vortex motion within the tube draws the sediment into it, and the wall velocities along the tube eventually eject the sediment at its discharge end. A properly designed vortex tube ejector can be more efficient than any other conventional ejector, with less water loss.

9.2.4 Settling basin

The settling basin is a device placed on the canal downstream of its head regulator for the removal of sediment load which cannot be trapped by the conventional excluders or ejectors. It consists of an enlarged section of the channel where the flow velocity is sufficiently low so that the fine sediment settles on the bed (Fig. 9.24). The settled sediment is removed by sluicing, flushing or dredging.

The following equation may be used to design a settling basin:

$$W = W_0 e^{-w_s x/q} \qquad (4.13)$$

where *W* is the weight of sediment leaving the basin, *W*0 is the weight of sediment entering the basin, *w*s is the fall velocity of a sediment particle (equation (8.18)), *q* is the discharge per metre width of the basin and *x* is the length of the settling basin. Alternatively

$$x = cD_s V/w_s \qquad (9.23)$$

where *D*s is the depth of the settling basin, *V* is the mean velocity in the basin, and *c* is the safety factor (1.5–2).

On the basis of the steady-state two-dimensional dispersion equation, the fraction of removal, *f*, at a distance *x* in a sedimentation tank of depth *y*, can be obtained (assuming isotropic dispersion) from

$$x/y = [-12V \log(1-f)]/(10w_s - U_*) \qquad (9.24)$$

where *U** is the shear velocity in the basin, given by equation (8.7).

Equation (9.24) gives reasonably satisfactory results when computing each fraction of removed sediment independently, if the concentration is small.

For a detailed discussion of sediment exclusion at inlets see Ortmanns (2006).

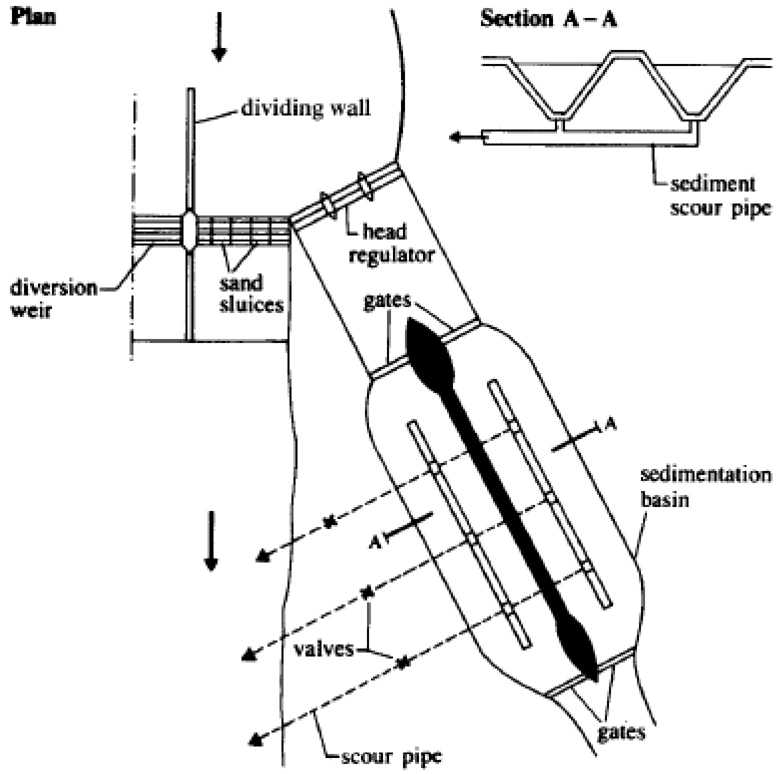


Fig. 9.24 Arrangement of settling basin

9.2.5 Canal outlets

The canal outlet is an intake structure for small canals, usually designed for proportional distribution of irrigation water supplies from the parent canal. These structures do not require any control works in the parent canal and function automatically with no manual control. Except for routine inspection they require very little maintenance (Mazumdar, 1983).

The flexibility of an outlet (*F*) is the ratio of the fractional change of discharge of the outlet (d*q*/*q*) to the fractional change of discharge of the parent (distribution) canal (d*Q*/*Q*). Thus,

$$F = (\mathrm{d}q/q)/(\mathrm{d}Q/Q) \qquad (9.25)$$

where q and Q are the outlet and parent canal discharges respectively, given by

$$q = kH^m, \qquad (9.26a)$$

$$Q = cD^n, \qquad (9.26b)$$

H is the head acting on the outlet and D is the depth of water in the distribution canal (Fig. 9.25).

Combining equations (9.25) and (9.26) gives

$$F = (m/n)(D/H)\mathrm{d}H/\mathrm{d}D. \tag{9.27a}$$

Since d*H*_d*D* (change in *D* results in an equal change in *H*), equation (9.27a) becomes

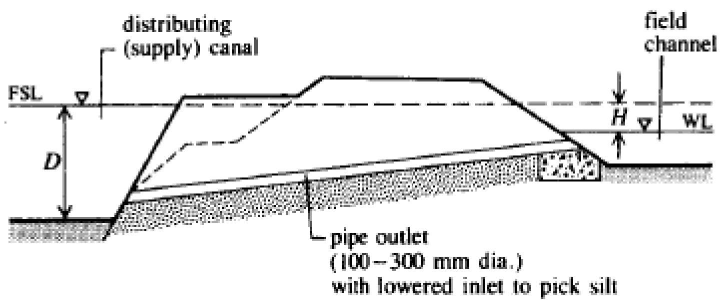
$$F = (m/n)(D/H). \tag{9.27b}$$

The design of a proportional outlet (*F*_1) is thus governed by the condition

$$H/D = m/n. \tag{9.28}$$

The ratio *H*/*D* is called the setting ratio of the outlet.

For an orifice or pipe-type proportional outlet (*m*_1/2) from a trapezoidal channel (*n*_5/3), the setting ratio *H*/*D* is 0.30, whereas for an open flume (Crump type, *m*_3/2) proportional outlet it is 0.90.



(H/D = 0.3 for proportional outlet)

Fig. 9.25 A submerged pipe outlet (non-modular type)

(a) Classification of outlets

1. *Non-modular type.* This is an outlet in which the discharge depends upon the difference in level (*H*) between the water levels in the distributing (supply) channel and the water course (field channel). A submerged pipe outlet (Fig. 9.25), or an orifice, is a common example of this type. Referring to the pipe outlet layout in

Fig. 9.25, any conventional pipe flow solution will give the outlet discharge, e.g. an iterative procedure combining Darcy–Weisbach and Colebrook–White equations.

2. *Semi-modular or flexible type.* The discharge through this type of outlet (semimodule) is only affected by the change in the water level of the distributing canal. A pipe outlet discharging freely, an open flume outlet and an adjustable orifice outlet are the common types of flexible outlet. As the discharge through this type of outlet is independent of the water level in the field

channel, interference by farmers is minimal. The pipe outlet is usually constructed with a flexibility F<1, so that the outlet discharge changes by a smaller percentage than the change in the distributing channel discharge. An open flume type outlet is shown in Fig. 9.26. The entrance is so designed that the flume takes a fair share of the silt. The discharge through this type of outlet (e.g. Crump) is given by

$$Q = CbH^{3/2}\ (\mathrm{m^3\,s^{-1}}) \qquad (9.29)$$

where b is the throat width (in m), H is the head over the crest (in m), and C equals 1.71 (theoretically 0.544g1/2; actual range 1.60–1.66).

An adjustable orifice semi-module (AOSM) type outlet, consisting of an orifice followed by a gradually expanding downstream flume, is shown in Fig. 9.27. This type of outlet is commonly used in the

Indian subcontinent, and is considered to be one of the best forms of outlet.

3. *Modular type.* This type of outlet (rigid or invariant module) delivers a constant discharge within set limits, irrespective of the water level fluctuations in the distributing channel and/or field channel. Gibb's module (Fig. 9.28) is one of the several types of rigid module outlet.

It has a semicircular eddy chamber in plan, connected by a rising inlet pipe. A free vortex flow is developed within the inlet pipe, thus creating a rise in the water level at the outer circumference of the eddy chamber (since Vr=constant for free vortex flow). The excessive energy of the entering flow (due to the increase in the head causing the flow) is dissipated by the baffle walls supported from the roof, thereby keeping the discharge constant over a wide range of variations in the head. An Italian design rigid module outlet is shown in Fig. 9.29. It consists of a cylindrical sleeve with circumferential apertures (which act as weirs), floating in a fixed outer cylinder. This ensures a constant head, causing a flow through the apertures irrespective of the water level in the distribution canal (Water and Water Engineering, 1956). The Neyrpic orifice type of rigid module outlet (French design) is shown in Fig. 9.30; this facilitates the drawing of an almost constant discharge over a wide range of water level fluctuations in the distribution channel. With the increase in head the contraction of the jet increases, thus offsetting any increase in the corresponding discharge. In order to cope with a much wider range of water-level fluctuations, a double baffle plate orificed module may be used. If the water-level fluctuations are beyond tolerable limits for constant flow in the off-taking canal, auxiliary equipment such as downstream and upstream level gates (Kraatz and Mahajan, 1975) must be installed in the distribution system.

10.3 Drop structures

10.3.1 Introduction

A drop (or fall) structure is a regulating structure which lowers the water level along its course. The slope of a canal is usually milder than the terrain slope as a result of which the canal in a cutting at its head works will soon outstrip the ground surface. In order to avoid excessive infilling the bed level of the downstream canal is lowered, the two reaches being connected by a suitable drop structure (Fig. 10.13).

The drop is located so that the fillings and cuttings of the canal are equalized as much as possible. Wherever possible, the drop structure may also be combined with a regulator or a bridge. The location of an offtake from the canal also influences the fall site, with offtakes located upstream of the fall structure.

Canal drops may also be utilized for hydropower development, using bulb- or propeller-type turbines. Large numbers of small and medium sized drops are desirable, especially where the existing power grids are far removed from the farms. Such a network of micro-installations is extremely helpful in pumping ground water, the operation of agricultural equipment, village industries, etc. However, the relative economy of providing a large number of small falls versus a small number of large falls must be considered. A small number of large falls may result in unbalanced earthwork but, on the other hand, some savings in the overall cost of the drop structures can be achieved.

Drops are usually provided with a low crest wall and are subdivided into the following types: (i) the vertical drop, (ii) the inclined drop and (iii) the piped drop.

The above classification covers only a part of the broad spectrum of drops, particularly if structures used in sewer design are included; a comprehensive survey of various types of drops has been provided, e.g. by Merlein, Kleinschroth and Valentin (2002); Hager (1999) includes the treatment of drop structures in his comprehensive coverage of wastewater structures and hydraulics.

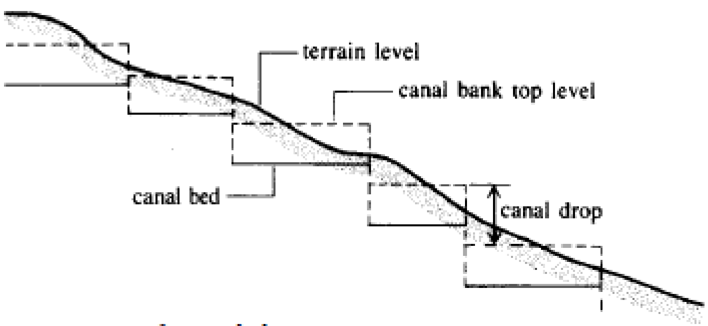


Fig. 10.13 Location of canal drops

10.3.2 Vertical drop structures

(a) Common (straight) drop

The common drop structure, in which the aerated free-falling nappe (modular flow) hits the downstream basin floor, and with turbulent circulation in the pool beneath the nappe contributing to energy dissipation, is shown in Fig. 10.14.

The following equations fix the geometry of the structure in a suitable form for steep slopes:

drop number, $D_r = q^2/g^{d3}$ (10.28)

where q is the discharge per metre width; basin length,

basin length, $L_B/d = 4.3D_r^{0.27} + L_j/d$; (10.29)

pool depth under nappe, $Y_P/d = D_r^{0.22}$; (10.30)

sequent depths, $y_1/d = 0.54D_r^{0.425}$; (10.31)

$y_2/d = 1.66D_r^{0.27}$; (10.32)

here d is the height of the drop crest above the basin floor and Lj the length of the jump.

A small upward step, h (around $0.5<h/y1<4$), at the end of the basin floor is desirable in order to localize the hydraulic jump formation. Forster and Skrinde (1950) developed design charts for the provision of such an abrupt rise. The USBR (Kraatz and Mahajan, 1975) impact block type basin also provides good energy dissipation under low heads, and is suitable if the tail water level (TWL) is greater than the sequent depth, $y2$. The following are the suggested dimensions of such a structure (Fig. 10.15):

basin length $L_B = L_d + 2.55y_c$; (10.33)

location of impact block, $L_d + 0.8y_c$; (10.34)

minimum TW depth, $y_2 \geq 2.15y_c$; (10.35)

impact block height, $0.8y_c$; (10.36)

width and spacing of impact block, $0.4y_c$; (10.37)

end sill height, $0.4y_c$; (10.38)

minimum side wall height, $y_2 + 0.85y_c$; (10.39)

here *y*c is the critical depth.

The values of *L*d can be obtained from Fig. 10.16.

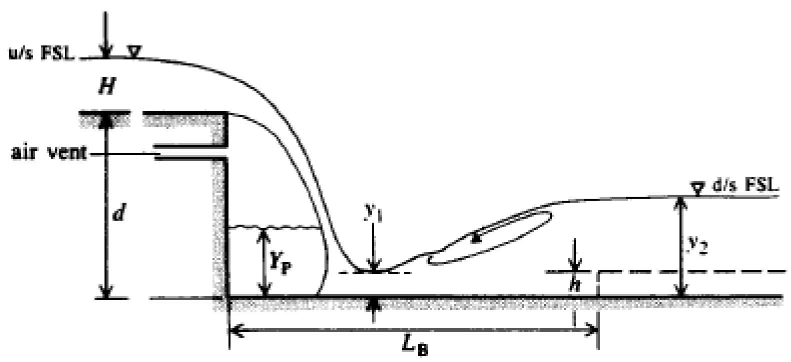


Fig. 10.14 Common drop structure (after Bos, 1976)

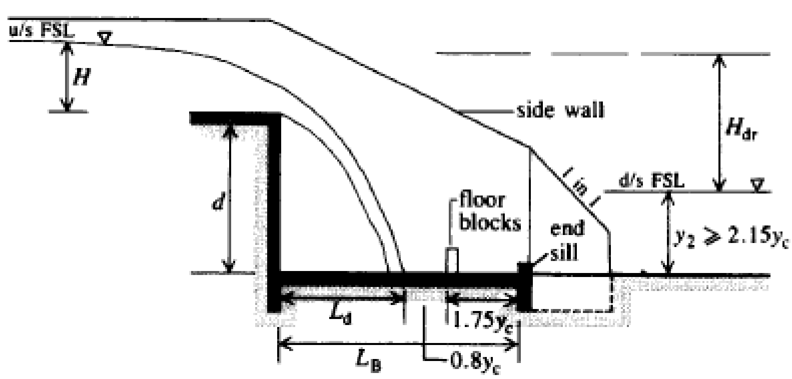


Fig. 10.15 Impact block type basin (after Bos, 1976)

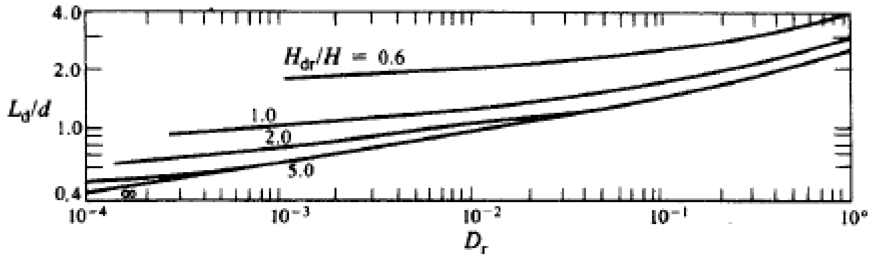


Fig. 10.16 Values of L_d/d (after Bos, 1976)

(b) Sarda-type fall (India)

This is a raised-crest fall with a vertical impact, consisting of a crest wall, upstream and downstream wing walls, an impervious floor and a cistern, and downstream bank and bed protection works (Fig. 10.17).

The crest design is carried out as follows. The crest length is normally kept equal to the bed width of the canal; however, an increase in length by an amount equal to the flow depth takes into account any future increase in discharge.

Fluming may be provided to reduce the cost of construction of the fall. A flumed fall with a fluming ratio equal to $2F1$, where $F1$ is the approach flow

Froude number, creates no choking upstream of the fall. A canal is not usually flumed beyond 50%. Whenever the canal is flumed, both upstream (contracting) and downstream (expanding) transitions have to be provided (Fig.10.3).

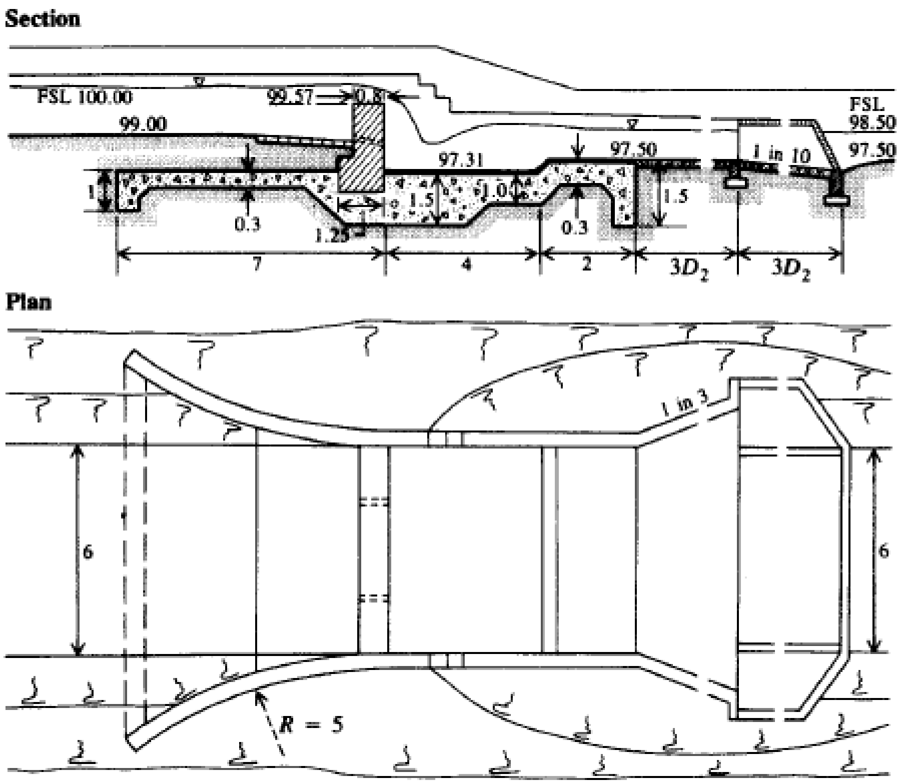


Fig. 10.17 Sarda fall layout (Worked example 10.7); dimensions in metres

The crest level must be so fixed that it does not create changes in upstream water levels (backwater or drawdown effects). If the reduced level (RL) of the full supply level (FSL) is *Y*, the RL of the total energy line (TEL) is

$$E = Y + V_a^2/2g \qquad (10.40)$$

where *V*a is the approach velocity.

If *L*e is the effective length of the crest, the head causing flow is given by the weir formula:

$$H = (Q/C_d L_e)^{2/3} \qquad (10.41)$$

where *Q* is the discharge and *C*d is the discharge coefficient of the crest.

Therefore, the RL of the crest is *E*-*H*.

Two types of crest are used (Fig. 10.18); the rectangular one for discharges up to 10m3 s_1 and the trapezoidal one for larger discharges (see Punmia and Lal, 1977).

The following are the design criteria established by extensive model studies at the Irrigation Research Institute in India.

1. For a rectangular crest,

top width, $B = 0.55d^{1/2}$ (m), (10.42)

base width, $B_1 = (H + d)/S_s$, (10.43)

where *S*s is the relative density of the crest material (for masonry, *S*s=2). The discharge is given by the following formula:

$$Q = 1.835LH^{3/2}(H/B)^{1/6}. \quad (10.44)$$

2. For a trapezoidal crest,

top width, $B = 0.55(H + d)^{1/2}$ (m). (10.45)

For the base width, *B*1, upstream and downstream slopes of around 1in 3 and 1 in 8 are usually recommended. The discharge is given by the following formula:

$$Q = 1.99LH^{3/2}(H/B)^{1/6}. \quad (10.46)$$

3. Design of cistern is as follows:

length, $L_c = 5(EH_{dr})^{1/2}$; (10.47)

depth, $d_c = \frac{1}{4}(EH_{dr})^{2/3}$. (10.48)

4. Minimum length of impervious floor downstream of the crest,

$$L_{bd} = 2(D_1 + 1.2) + H_{dr}. \quad (10.49)$$

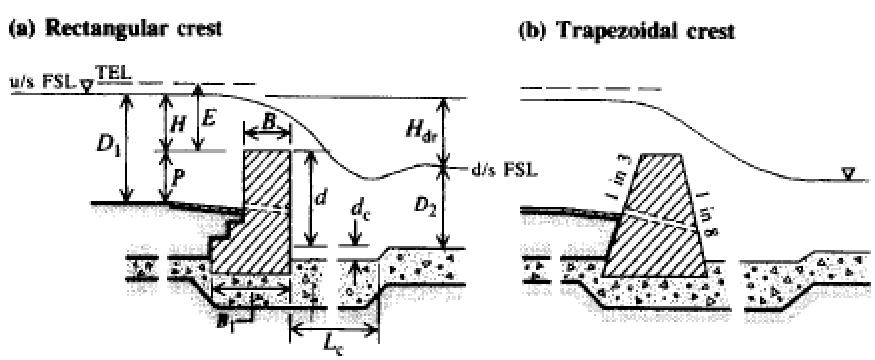


Fig. 10.18 Sarda fall crests

(c) YMGT-type drop (Japan)

This type of drop is generally used in flumed sections suitable for small canals, field channels, etc., with discharges up to 1m3 s_1 (Fig. 10.19). The following are the recommended design criteria:

1. Sill height, P varies from 0.06 m to 0.14 m with the unit discharge q between 0.2 and 1.0m^3/s/m;

2. depth of cistern, $d_c = 1/2(E_c H_{dr})^{1/2}$; (10.50)
3. length of cistern, $L_c = 2.5 L_d$, (10.51)

where Ld=Ld1+Ld2 and

$$L_{d1}/E_c = 1.155[(P'/E_c) + 0.33]^{1/2}, \quad (10.52)$$

$$L_{d2} = (D_2 + d_c)\cot\alpha, \quad (10.53)$$

$$\cot\alpha = y_c/L_{d1}. \quad (10.54)$$

Alternatively, the recommendations of the IRI, India (previous section) may also be adopted.

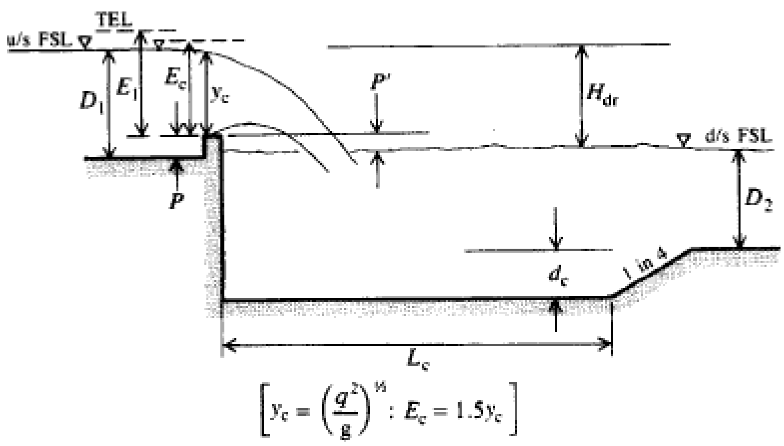


Fig. 10.19 YMGT-type drop, Japan (Kraatz and Mahajan, 1975)

(d) Rectangular weir drop with raised crest (France) SOGREAH (Kraatz and Mahajan, 1975) have developed a simple structure suitable for vertical drops of up to 7m (for channel bed widths of 0.2–1 m with flow depths (at FSL) of 0.1–0.7 m): Fig. 10.20 shows its design details.

1. For the design of crest,

$$\text{discharge,} \quad Q = CL(2g)^{1/2}H^{3/2}, \quad (9.29)$$

where C=0.36 for the vertical upstream face of the crest wall and 0.40 for the rounded upstream face (5–10 cm radius). The crest length, L=LB-0.10 m for a trapezoidal channel and is B1 (the bed width) for rectangular channels.

2. For the design of cistern,

$$\text{volume of basin,} \quad V = QH_{dr}/150 \ (\text{m}^3), \quad (10.55)$$

width of basin, $W_B = V/[L_B(D_2 + d_c)]$,

where the depth of the basin, d_c=0.1-0.3m.

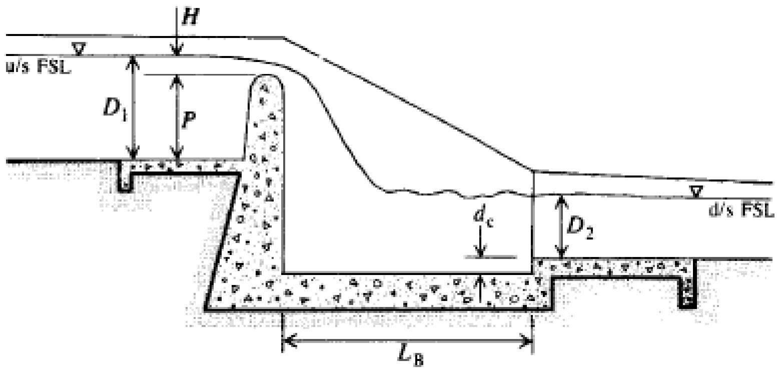


Fig. 10.20 Rectangular weir drop with raised crest, France (Kraatz and Mahajan, 1975)

10.3.3 Inclined drops or chutes

(a) Common chute

This type of drop has a sloping downstream face (between 1/4 and 1/6, called a glacis) followed by any conventional type of low-head stilling basin; e.g. SAF or USBR type III (Chapter 5). The schematic description of a glacis-type fall with a USBR type III stilling basin, recommended for a wide range of discharges and drop heights, is shown in Fig. 10.21.

(b) Rapid fall type inclined drop (India)

This type of fall is cheap in areas where stone is easily available, and is used for small discharges of up to 0.75m3 s_1 with falls of up to 1.5 m. It consists of a glacis sloping between 1 in 10 and 1 in 20. Such a long glacis assists in the formation of the hydraulic jump, and the gentle slope makes the uninterrupted navigation of small vessels (timber traffic, for example) possible.

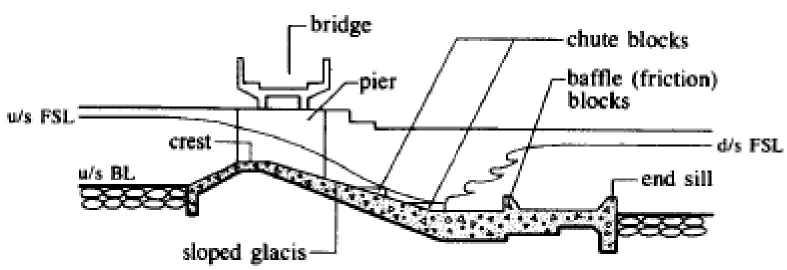


Fig. 10.21 Sloping glacis type fall with USBR type III stilling basin

(c) Stepped or cascade-type fall

This consists of stone-pitched floors between a series of weir blocks which act as check dams and are used in canals of small discharges; e.g. the tail of a main canal escape. A schematic diagram of this type of fall is shown in Fig. 10.22.

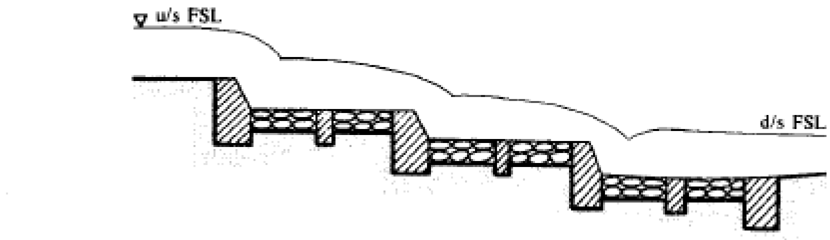


Fig. 10.22 Stepped or cascade-type fall

10.3.4 Piped drops

A piped drop is the most economical structure compared with an inclined drop for small discharges of up to 50 l /s. It is usually equipped with a check gate at its upstream end, and a screen (debris barrier) is installed to prevent the fouling of the entrance.

(a) Well drop structure

The well drop (Fig. 10.23) consists of a rectangular well and a pipeline followed by a downstream apron. Most of the energy is dissipated in the well, and this type of drop is suitable for low discharges (up to 50 l /s) and high drops (2–3 m), and is used in tail escapes of small channels.

(b) Pipe fall

This is an economical structure generally used in small channels. It consists of a pipeline (precast concrete) which may sometimes be inclined sharply downwards (USBR and USSR practice) to cope with large drops.

However, an appropriate energy dissipater (e.g. a stilling basin with an end sill) must be provided at the downstream end of the pipeline.

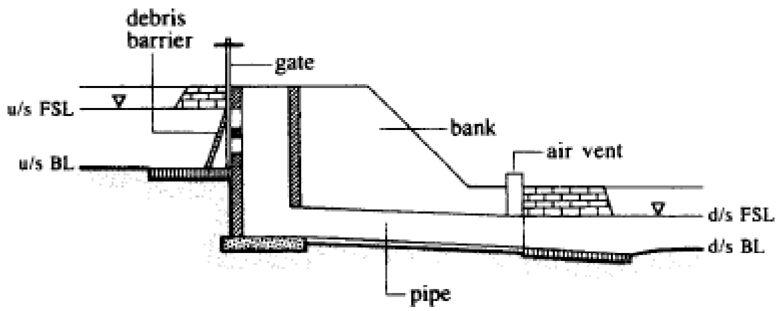


Fig. 10.23 Well drop structure

10.3.5 Farm drop structures

Farm channel drops are basically of the same type and function as those in distribution canals, the only differences being that they are smaller and their construction is simpler.

The notch fall type of farm drop structure (precast concrete or timber) consists of a (most commonly) trapezoidal notch in a crested wall across the canal, with the provision of appropriate energy-dissipation devices downstream of the fall. It can also be used as a discharge measuring structure.

The details of a concrete check drop with a rectangular opening, widely used in the USA, are shown in Fig. 10.24. Up to discharges of about $0.5 m^3/s$, the drop in the downstream floor level (*C*) is recommended to be around 0.2 m and the length of the apron (*L*) between 0.75 m and 1.8m over a range of drop (*D*) values of 0.3–0.9m.

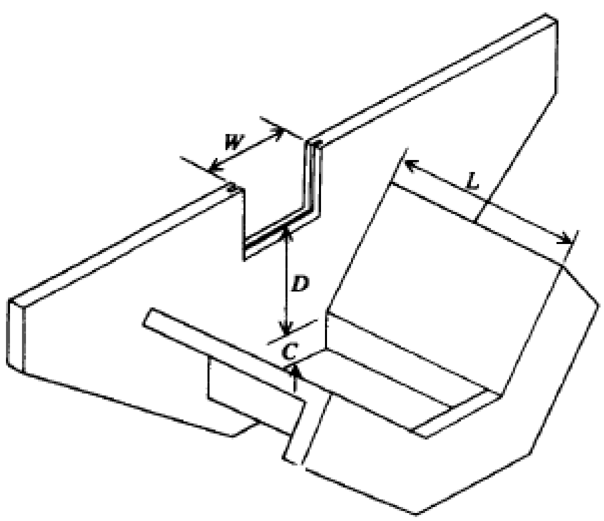


Fig. 10.24 Notch fall: concrete check drop (USA)