SEEPAGE AND STABILITY ANALYSIS OF EARTH DAMS

by

Khosrow Mahgerefteh

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APPROVED:

S. Sture, Chairman

D. N. Contractor

T. Kuppusamy

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To my parents

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CHAPTER 1

INTRODUCTION

1.1 Earth Dams--Historical Background

Earth dams have been constructed since the earliest recorded history of civilization and they are among the largest man made constructed facilities, whose general features and purpose have not changed markedly with time. They are also among the most common and economical types of structures with respect to cost per unit placed mass, constructed on foundations of geologic materials, particularly if the earth material for the embankment is readily accessible.

An earth dam is a large structure containing much mass and its stability is mainly due to gravitationally induced actions transmitted to the foundation. An earth dam can be wide or narrow depending on the topography and the availability of borrow construction material.

Concrete arch dams, on the other hand are only feasible structures in those instances where relatively narrow canyons or valleys in conjunction with solid and competent rock masses, are in the proximity of construction material sources. Concrete arch dams which are slender structures that get their strength and stiffness from the reinforced concrete's material properties and the structure's geometry, can only be constructed when the economical and environmental conditions are suitable. This is seldom the case in most parts of the world.

Earth dams are built to store water for domestic use, irrigation, to prevent soil erosion, and as a means for flood control.

Historical records indicate that earth embankments ranging from 15 ft to 30 ft high were constructed as early as 500 B.C. [Ref. 22]. Between 500 A.D. and 1500 A.D., numerous earth dams were constructed in different parts of the world, and the largest of these, Madduk-Masur Dam, 108 ft high was completed in India about 1500 A.D. Because the dam lacked a spillway, it overtopped and failed [Ref. 22]. In 1789, Estrecho de Rients was complted in Spain [Ref. 5]. It had a height of 150 ft, and it took 34 years to build the dam. Three years later, after its reservoir had been filled, the dam was breached, because of insufficient spillway capacity.

Between 1920 and 1930, a large number of dams failed. Three of these failures occurred in Britain and one in Italy. The cause of these failures were attributed to high shear stresses, and bad workmanship. Several dams have experienced more than one failure in its operational life. In Algeria the Habra Dam failed in 1927. This dam had already experienced a collapse due to distressed hydraulic conditions in 1881 [Ref. 5]. The causes of failure were due to overtopping in both instances. Failure has also been caused by problems in the foundation. The Francis Dam near Los Angeles was completely wiped out in 1928 when its foundation was weakened and washed away, because of high seepage gradients [Ref. 11].

Morise (1939) stated that, "There is probably no type of structure built by man which offers so great a potential hazard to life and property as large dams and reservoirs upstream from a heavily populated area." This statement has probably greater meaning today because many high density population centers have grown up close to large dam facilities. This problem was brought to the public attention in 1889 when a dam across the Conemaugh River in Pennsylvania breached after being overtopped after a heavy rainfall. As a result of this failure, many thousands of people lost their lives and much property was destroyed in the town of Johnstown [Ref. 8].

Catastrophic dam failures in more recent times occurred even in carefully designed and constructed dams. The Teton Dam [Ref. 2] failure in 1976 resulted in the loss of few lives but much property. The Morvi Dam (India) failure in August 1979 resulted in the loss of several thousand lives and the complete erasure of a town. The cause of the latter was due to heavy rainfall (which resulted in overtopping), and the cause of failure in the former has been attributed to hydro-fracturing of the rock beneath the dam foundation.

During the first half of the twentieth century dam design and construction practices in most countries went through four major developments:

1. Dams became generally larger.
2. Construction techniques were greatly improved in part by the introduction of heavy earth moving equipment.
3. Rational methods of analysis led to a more complete understanding of the structural behavior of dams and these developments in turn resulted in better design procedures.

4. Improved quality control and construction organization resulted in more reliable dam structures; aspects which were virtually absent earlier.

It was approximately around 1930 that all of these factors were considered and debated publicly for the first time. As a result of these developments, many large dams have successfully been built since then, and the failure rate has dropped sharply. For instance, the Windsor Dam in Massachusetts was completed in 1940, to a height of 295 ft, the Anderson Ranch Dam in Idaho was completed in 1950, to a height of 455 ft, the Trinity Dam in California was completed in 1960 to a height of 537 ft [Ref. 13], and more recently constructed dams approach 1000 ft [Ref. 8].

It is remarkable that these developments have been achieved with a minimum of severe problems. However, these large structures are continuously monitored for safety.

Sherard et al. (1963) have tabulated and discussed more than 200 dam failures [Ref. 7]. They mention that the most common cause of failure could be attributed to environmental causes such as heavy rainfalls resulting in overtopping or problems related to the foundation material.

During the planning and design stages of an earth dam, the engineer must make every effort to eliminate the possibility or sources for unsatisfactory performance or total failure of the structure. The design, analysis and construction of an earth dam are a synthesis of numerous factors. The most important elements and features will be discussed in the following sections and later chapters.

1.2 Fundamental Steps in the Design of Earth Dams

The following criteria must be followed during the planning and construction phases in order to obtain a safe and economical earth structure [Ref. 11, 12, 23].

1. Planning and execution phases of the exploration of foundations, abutments, and adjacent faults must be carried out in detail, and potential sources of borrow materials for construction of the dam must be studied carefully. Field investigation should always be conducted before any design is considered.
2. Study relevant environmental, topographical and other conditions influencing the design:

   Climate and particular seasonal activity

   Surface and subsurface geology

   Shape and size of valley

   River diversion problems

   Problems associated with wave action during filling and operation

   Available construction time

   Function of reservoir

   Seismic activity

   Ground water conditions

3. Selection of possible trial designs.
4. Analysis of safety and deformational behavior of the trial designs.
5. Modification of the design based on given specifications or environmental factors.
6. Estimation of cost and time schedules for given design and associated feasibility investigation.
7. Final selection of the design offering the best combination of economy and safety.

These criteria and schedules are generally adhered to, and some of them will be discussed in greater detail later.

The next section deals with particular problems associated with dam behavior and safety.

1.3 Factors Related to Analysis, Design, and Construction Requiring Further Investigation

During the last decade extensive studies have been conducted relating to factors that influence the safety of dams [Ref. 8, 16, 12], because failures do still occur in spite of all possible modern precautions and construction techniques. The causes for many recent and past failures still elude investigative committees and designers. Some typical features follow:

1. Strength stiffness and volume change behavior of soils under high confining pressures.

2. Strength of compacted clays for end of construction and long term conditions.
3. Strength of compacted clay for long term steady seepage conditions.
4. Strength of compacted clay for rapid drawn down conditions.
5. Compaction requirements for coarse gravels and rockfills.
6. Cracking within embankments from differential settlement and extensional strains.
7. Prediction of pore pressures in compacted cohesive and cohesionless soils.
8. Stress and deformation prediction and measurements in embankments during construction and operation.
9. Dynamic behavior of embankments for use in earthquake analysis and design.
10. Slope protection for erosion control.
11. Behavior during drawdown situations.
12. Freeboard, spillway, and ancillary dam facility requirements.

All the essential elements, some of which are listed above, related to construction, design, and analysis of earth dams were established by approximately 1940, but improvements in theory and application are still being elaborated on, since many questions remain unanswered, exemplified in the recent failures discussed earlier.

The cross section of a typical dam and some of the related features are shown in Fig. 1.1a-c. A typical earth dam cross section

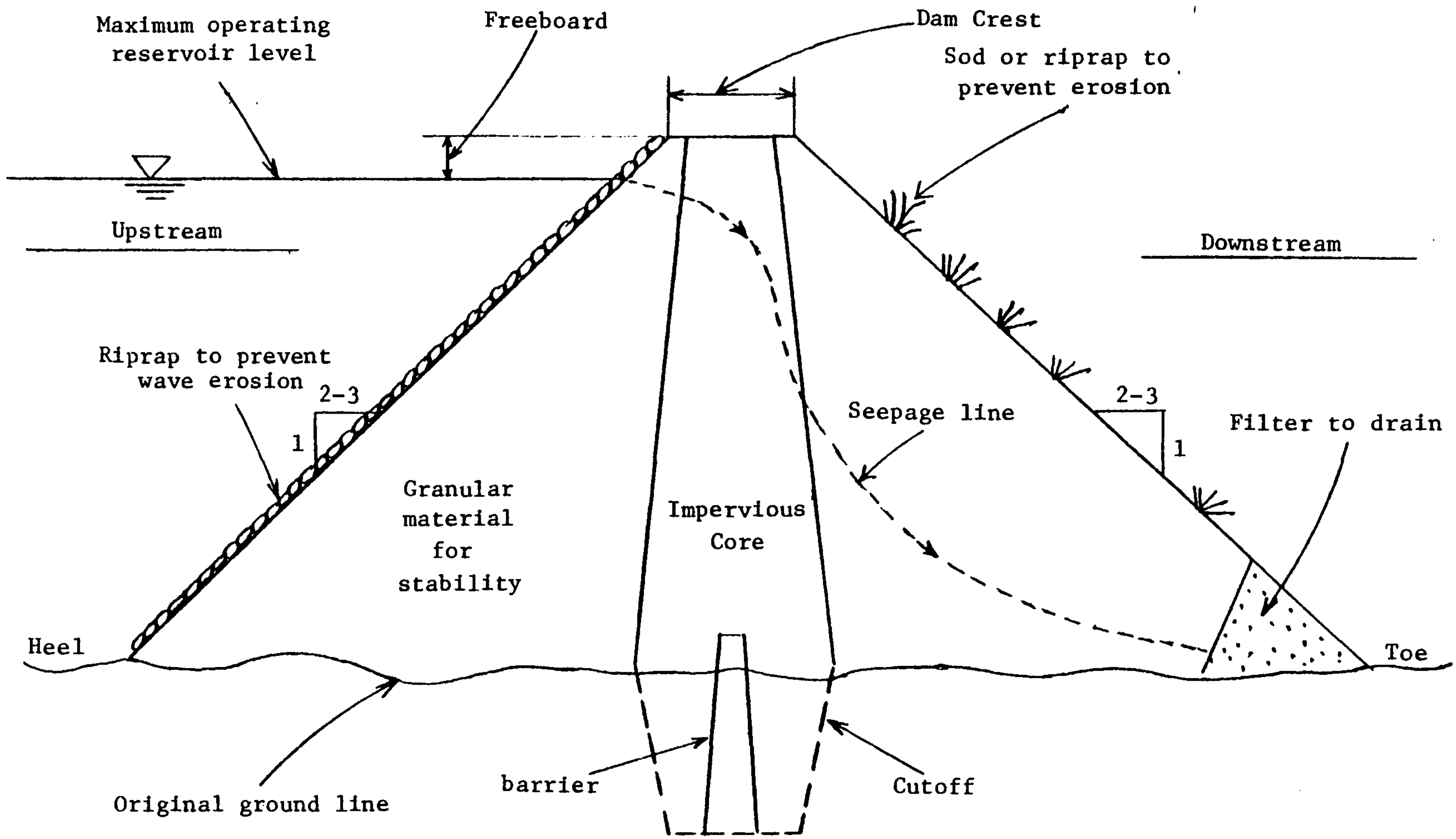


Fig. 1.1a· Typical cross section of dam.

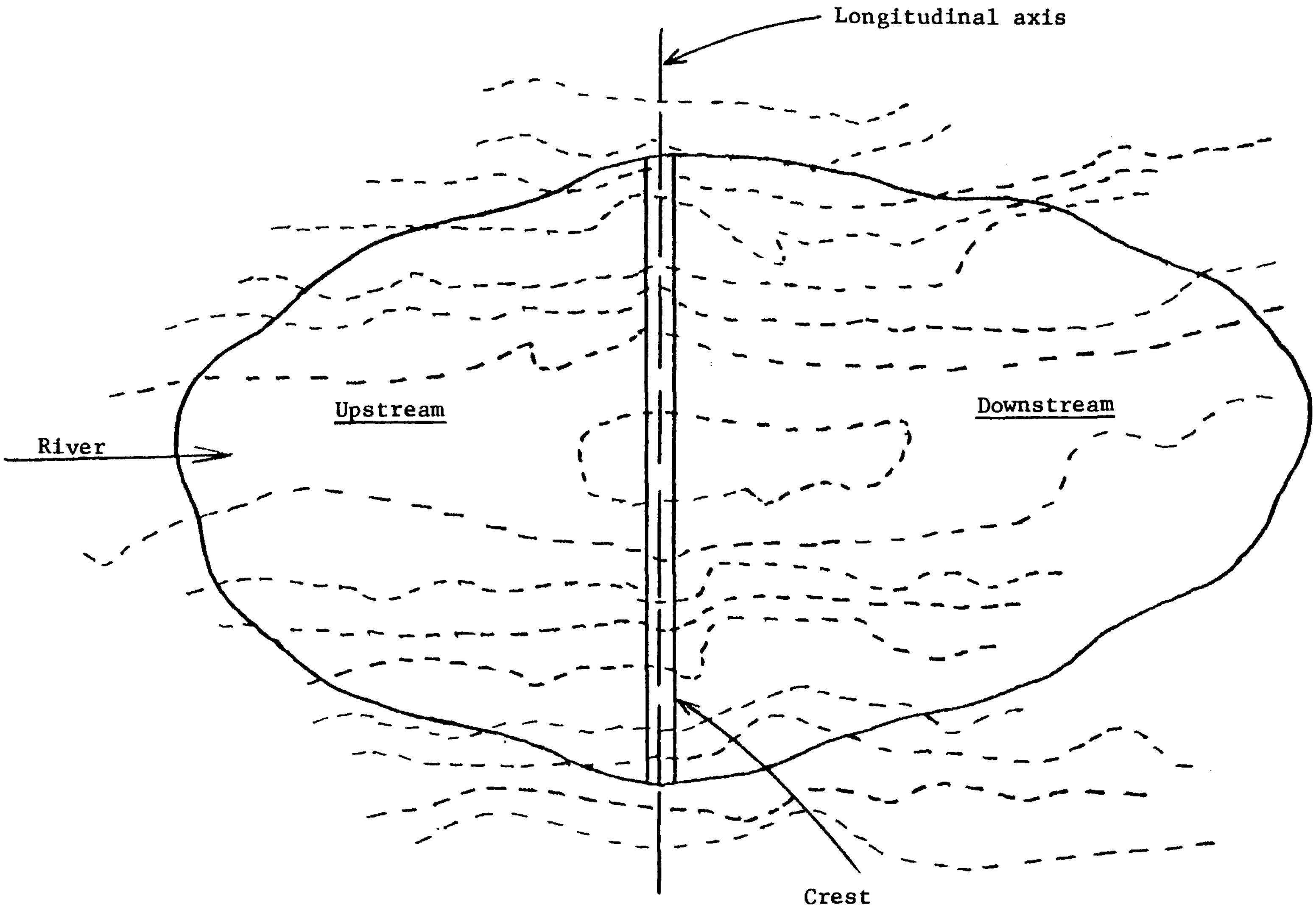


Fig. 1.1b Plan view of dam.

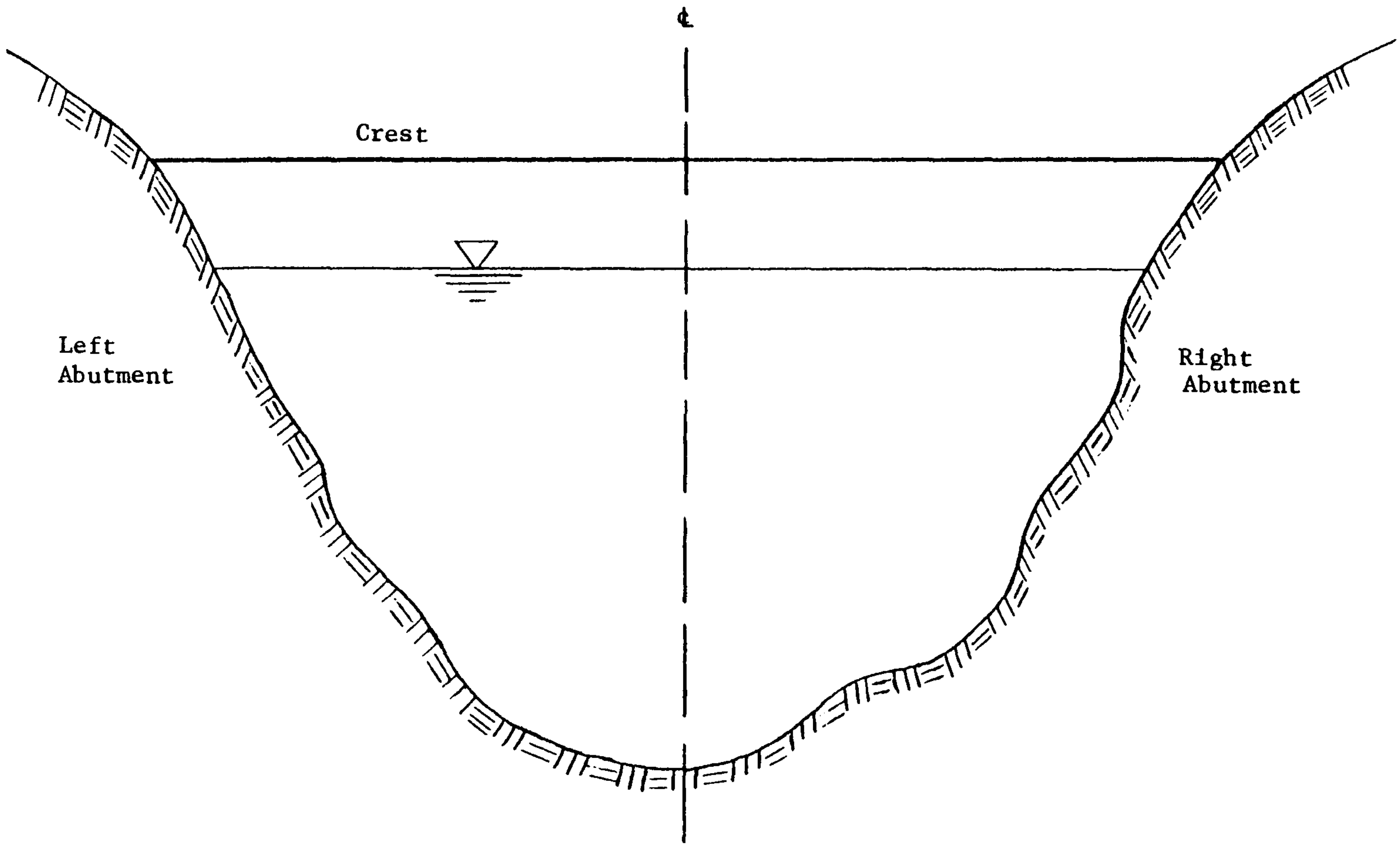


Fig. 1.1c Profile of valley along the axis of the dam.

has the shape of a trapezoid. Its upstream and downstream surface slopes are seldom steeper than 2:1. The width of the dam's crest can vary depending on its function. For instance; many highways, parks, and buildings are located on dam crests. Impervious cores which serve to stabilize or control seepage through the dam are usually located close to the center of the dam, but their shapes are often arbitrary. The impervious core is discussed in greater detail later. Filters are often located near the toe region of dams and often inside in order to reduce or control rate of seepage. Cutoff trenches and barriers are also used to control seepage and they do also serve as structural stabilizing elements. Riprap in the form of crushed rock and cultivated carpets of sod serve to reduce erosion.

In designing and constructing earth dams the following features should be emphasized, because they are of a general nature and of great importance.

1. Compaction of the earth fill and the core should be made to maximum density at optimum moisture, and the core is frequently compacted slightly wet of optimum.
2. The foundation material should never consist of soft soil or peat, and the embankment soil is usually compacted in 6 in. layers or fills with power equipment.
3. The riprap or surface cover used to protect the upstream slopes of a dam against destructive wave action or erosion, can range from boulders or concrete pavement to thick layers of asphaltic concrete or flexible sheets of plastic.

4. Sod blends in with the environment as a camouflage, but other surface covers such as concrete slabs, and riprap are used as well.

The proper placement and compaction of earth materials are of great importance. This was not fully appreciated in the past, although it was often achieved to some extent during the normal process of transporting and depositing the material. The sheeps foot roller which is a heavy compaction facility, came into use at the beginning of the twentieth century, and between 1930 and 1950 it was the standard compaction device. After 1950 rubber-typed rollers have replaced or complemented the sheeps foot devices, because they have often been found to be more economical or easier to operate.

Earth dams may be constructed of almost any material with primitive construction equipment and practices or by technically sophisticated means. Recently built large dams in the Third-world have been erected by hundreds of thousands of workers with pit and shovel. On the other hand, dams in the U.S. and Europe are constructed by modern equipment and only hundreds of workers. Earth dams have successfully been constructed from boulders, sand, silt, rock flour, clay and general land fill waste. If a quantity of pervious materials such as sand and gravel is available and clayey material must be imported, the dam would have a small impervious clay core. The material available locally would always be making up the bulk of the dam. Concrete was often used in the past for impervious cores, but it would not provide the flexibility of clay material needed during dam movements in the course of filling or seasonal changes. If pervious materials in appreciable quantities are not available, then clayey material can be used for the primary construction element with underdrains of imported sand or gravel located near the downstream toe to collect seepage and reduce pore pressure build-up. This is discussed in further detail in Chapters 2 and 3.

In 1950 Casagrande recognized the desirability of using more plastic clays in the core in order to minimize the possibility of cracking and subsequent erosion [Ref. 4]. The flexibility of the core can also be achieved by compacting clay wet of optimum. A study of this problem was also performed by Leonards and Narain [Ref. 8]. They conducted laboratory beam flexure tests on core material from five dams. The results show that the tensile strain could cause cracking and the materials ability to sustain tensile strain is related to the plasticity, the moisture content, and density characteristics of the soil. Table 1.1 shows typical laboratory test data of soil used in the five dams. Each different material was compacted at optimum moisture content and density. Leonards and Narain also conducted other beam flexure test on highly plastic clay [Ref. 8]. The clay was compacted to moisture contents ranging from dry to 3 percent wet of optimum. From both experiments they concluded that highly plastic clays are more flexible and will minimize the possibility of cracking during tensile straining and localized shear.

TABLE 1.1

Laboratory test data of soil used in the five dams [Ref. 8].

Type of Material	Tensile Strain, %	Time of Tests
SM (silty sand)	0.19	4 weeks
SC (clayey sand)	0.17	4 weeks
SM	0.24	4 weeks
SM	0.07	2 days
SM	0.24	4 weeks

If some of the materials to be used were not flexible, the designer can establish different design criteria for the placement of the materials in order to reduce the possibility of crack development. The proper sequence and timing of construction that allows a compressible foundation to consolidate prior to placing the next lift or fill adjacent to or above stiffer or softer geologic foundation regions should also be investigated.

Since the failure of dams have serious consequences for life and property, the factors discussed previously and many other elements and schemes not elaborated on here must be considered in the analysis, design and construction of dams. The knowledge of mechanics and engineering techniques as well as good judgment must be combined to develop a good design. The construction procedures should be carefully supervised and quality control should be performed frequently.

1.4 Instrumentation

Instrumentation is used to monitor the performance of dams. Different types of instruments are installed inside and on the surface of many dams to measure horizontal displacements, settlements, strains, stresses, and pore water pressures. These instruments will provide quantitative and qualitative data concerning the behavior of the dam during various seasonal events, earthquakes, and sudden drawdowns. The results of these measurements will provide a better understanding of earth dams performance and consequently these measurements should be used to evaluate the safety of the structure during its operation. Detailed information on types, installation, and operation of instrumentation is given in many standard texts [Ref. 7, 18].

1.5 Scope of Work

The purpose of this report is to present a study into the behavior of earth dams. The specific problems which are to be discussed can be divided into three different categories; a) estimation of the quantity of seepage; b) methods for controlling seepage; c) stability analysis of embankments.

The quantity of seepage through earth structures is based on fundamental relationships such as Darcy's Law. The laws and principles related to conductance of water in porous media are discussed in Chapter 2. The governing equation for steady state seepage is also discussed in Chapter 2. The case of flow in nonhomogenous and anisotropic soils are discussed. The flow net construction solution scheme is described and it was used to determine the path and the rate of seepage through an earth mass.

Percolation of water through earth dams does frequently create serious problems, and in severe cases it can lead to total structural failure. In Chapter 3, the consequences of seepage in dams is discussed. Different methods are described for controlling the quantity of seepage.

Chapter 4 deals with the stability of earth embankments. The stability of earth slopes are considered under the assumption that slip occurs along a distinct and continuous surface. The factor of safety against stability failure is found. Four different analysis schemes are discussed. The shear strength on the potential sliding surface which is necessary to prevent movement, the resisting force and other forces are computed by static equilibrium conditions and by a strength criterion. The four different stability analysis schemes were compared, the differences, and similarities of the results were obtained and discussed.

CHAPTER 2

ANALYSIS OF SEEPAGE THROUGH POROUS MEDIA

2.1 General

In nearly all earth dams, there is some amount of water seeping through the embankment and its foundation. The flow of water through the earth mass has been the main cause of failure for many earth structures.

A method for analysis and a description of seepage phenomena in porous media such as soil were first discussed by Darcy (1856) [Ref. 26]. Darcy carried out experimental and theoretical investigations on the flow of water through pipes filled with sand, and he established an equation characterizing this movement. This equation is discussed in section 2.2.

Terzaghi (1925) showed that Darcy's Law is also valid for seepage through very fine grained soils such as clays. Terzaghi conducted further investigations (1930) emphasizing the importance of the seepage forces created within earth dams, due to the percolation of water through them [Ref. 5].

In 1937 Casagrande published a comprehensive paper on seepage through dams in which he demonstrated that Darcy's Law, which describes the relationship between velocity and total head gradient and the Laplace equation, which describes the distribution of the total head in the seepage domain, were the essential bases of seepage studies

[Ref. 7]. From his work subsequent developments and refinements have originated. Finite difference, finite element, electrical and other analogue models, and Forchheimer graphical techniques have later seen wide use in the analysis of two and three dimensional seepage problems [Ref. 1, 3, 6].

2.2 Darcy's Law

According to Darcy's empirical law, the velocity of the fluid flowing through a porous medium is proportional to the hydraulic gradient,

$$v = Ki = -K \frac{dh}{dL} \qquad (2.1)$$

where

i is the hydraulic gradient

K is the coefficient of permeability

h is the piezometric (total) head

L is the directed distance

The equations characterizing the velocities in three dimensions can be written in a similar way as,

$$\begin{aligned} v_x &= -K_x \frac{dh}{dx} \\ v_y &= -K_y \frac{dh}{dy} \qquad (2.2) \\ v_z &= -K_z \frac{dh}{dz} \end{aligned}$$

where

K_x, K_y, K_z are the coefficients of permeability in X, Y, Z directions respectively.

The validity of Darcy's Law has been confirmed by many experiments [Ref. 10]. According to Reynolds (1883), Darcy's Law is applicable whenever the flow through the pipes is laminar and the pore space is continuous [Ref. 10].

2.3 Coefficient of Permeability (K)

The coefficient of permeability can be expressed as the ease with which water passes through a material, and it has the dimension of velocity. It has been observed that the magnitude of the permeability coefficient depends on the viscosity of water, which in turn is a function of the temperature, and on the shape of the conduits through which the water flows.

In 1937 Muskat [Ref. 10] obtained a relationship between physical permeability (K_o) and the coefficient of permeability (K), given as:

$$K = K_o \frac{\gamma_w}{\mu} \tag{2.3a}$$

where

K_o = physical permeability; a constant for any permeable material at a given porosity or void ratio, and it is independent of the properties of the percolating fluid

γ_w = unit weight of water

μ = kinematic viscosity.

It should be noted that the real seepage velocity, V_s, is related to the model velocity (V), through the porosity (n),

$$V_s = \frac{V}{n} \tag{2.3b}$$

Substituting equation (2.3a) into Darcy's equation, we obtain:

$$v = -K_o \frac{\gamma_w}{\mu} \frac{dh}{dL} \tag{2.4}$$

which shows that the discharge velocity is inversely proportional to the viscosity of the fluid. The coefficient of permeability for a given soil can be determined from laboratory tests. Some typical values for the coefficient of permeability of soils are given in Table 2.1 [Ref. 10].

2.4 Two-Dimensional Flow

In reality, all flow systems are three dimensional. However, in many problems the features of the ground water movement are planar, with motion being the same in parallel planes. Structurally critical sections of dams are usually also the ones conducting the largest mass of seepage, and the water movement along a longitudinal segment near these deepest sections are also usually planar, as illustrated in Fig. 2.1.

2.4.1 Equation of Continuity and Laplaces Equation

Considering water is percolating through a homogeneous soil in such a manner that the voids of the soil are completely filled with

TABLE 2.1

Values of coefficient of permeability (K) for various soils [Ref. 10].

Soil Type	Coefficient of Permeability (K) (in/sec)
Clean gravel	1.0 and greater
Clean sand (coarse)	1.0-0.01
Sand (mixture)	0.01-0.005
Fine sand	0.005-0.001
Silty sand	0.002-0.0001
Silt	0.0005-0.00001
Clay	0.000001 and smaller

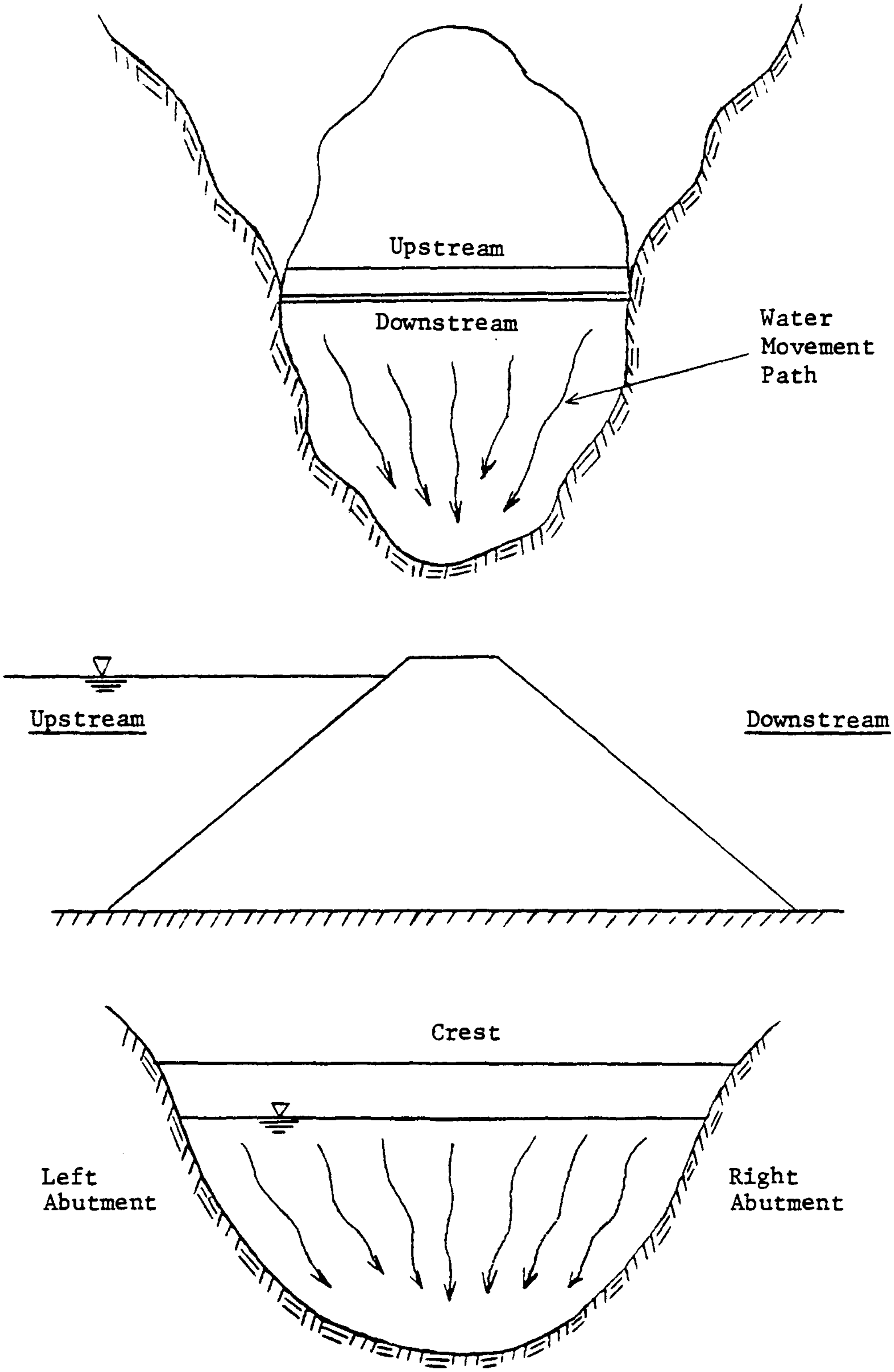


Fig. 2.1 Two dimensional flow situations.

water, then the quantity of water entering from several directions into a small element of soil must be equal to the amount of water flowing out from the other face of the elemental volume during any given unit of time. Fig. 2.2 shows a small elemental prism of soil having the dimensions of dx, dy, and dz. The following expression can be written for masses per time unit entering and exiting the unit volume,

$$\left(\frac{dm}{dt}\right)_{in} = \rho_{in}\,(v_x\,dy\,dz + v_y\,dx\,dz + v_z\,dx\,dy)$$

$$\begin{aligned}\left(\frac{dm}{dt}\right)_{out} = \rho_{out}\,\Big[&\left(v_x\,dy\,dz + \frac{\partial v_x}{\partial x}\,dx\,dy\,dz\right)\\ &+ \left(v_y\,dx\,dz + \frac{\partial v_y}{\partial y}\,dx\,dy\,dz\right)\\ &+ \left(v_z\,dx\,dy + \frac{\partial v_z}{\partial z}\,dx\,dy\,dz\right)\Big]\end{aligned}$$

The equation of continuity is established by equating the above expressions, assuming the mass density (ρ) to be constant and simplifying the resulting terms,

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z} = 0 \qquad (2.5)$$

Combining Darcy's equation (2.2) and equation (2.5), the governing equation for incompressible flow in a porous medium becomes:

$$K_x\,\frac{\partial^2 h}{\partial x^2} + K_y\,\frac{\partial^2 h}{\partial y^2} + K_z\,\frac{\partial^2 h}{\partial z^2} = 0 \qquad (2.6)$$

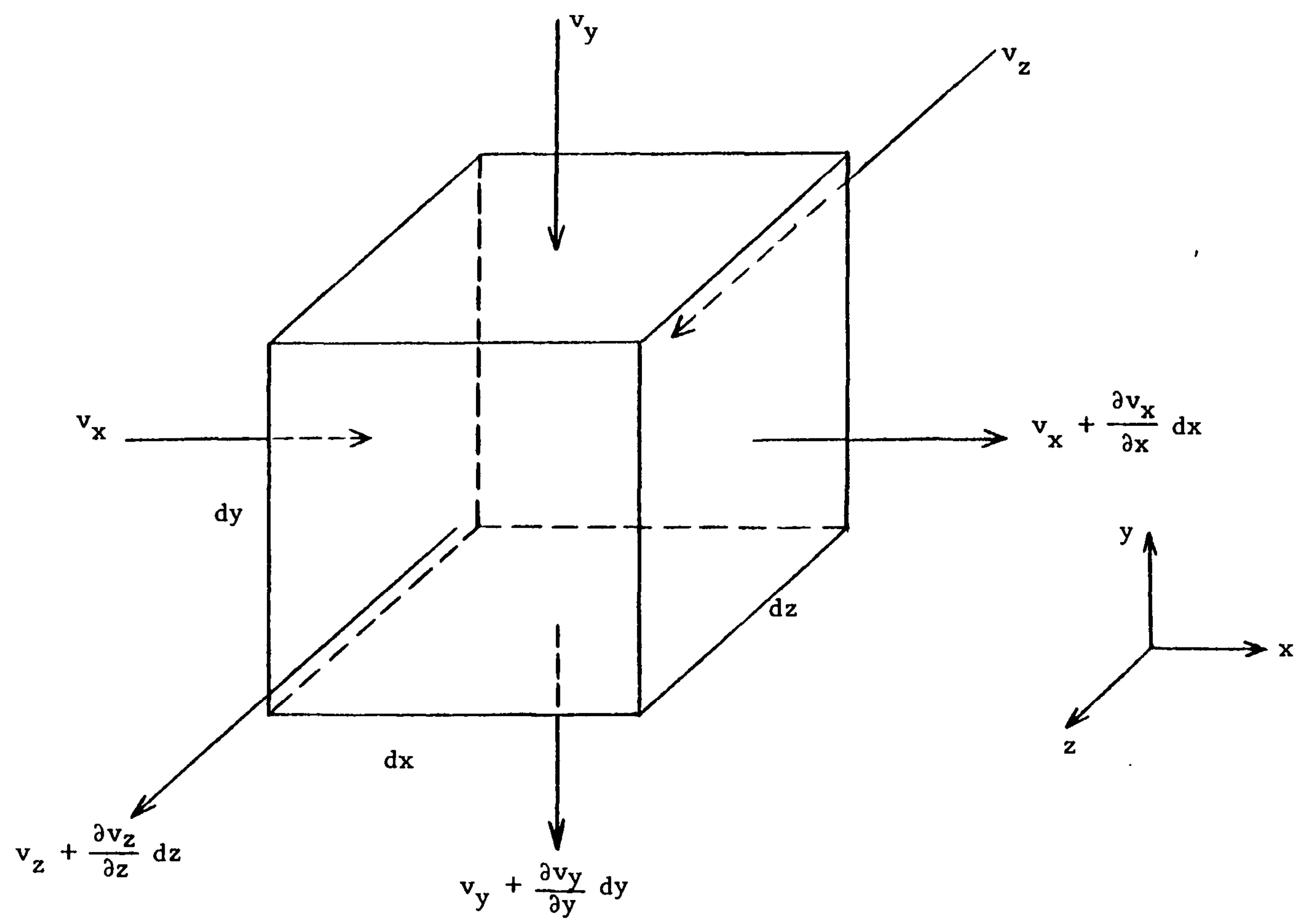


Fig. 2.2 Flow in and out of an element of soil.

For a homogeneous and isotropic material ($K_x = K_y = K_z$) equation (2.6) becomes:

$$\nabla^2 h = \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \tag{2.7}$$

which is the Laplace equation for steady state flow. For general seepage problems of practical interest, the flow is considered to be two dimensional as discussed earlier. Therefore, the Laplace's equation can be expressed as:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \tag{2.8}$$

The solution of this equation in an isotropic and homogeneous domain is represented by two families of curves intersecting at right angles. These curves are known as the flow lines which define the direction of flow, and the equipotential lines, which are contours of equal heads consisting of pressure and elevation heads. The entire pattern is known as the flow net and it can be obtained analytically or numerically by techniques outlined later. A flow net solution is shown in Fig. 2.3. Some of the features and properties of the flow net subjected to conditions mentioned in Section 2.5 can be summarized as follows:

1. The water flowing between any two adjacent flow lines is the same throughout the flow net.

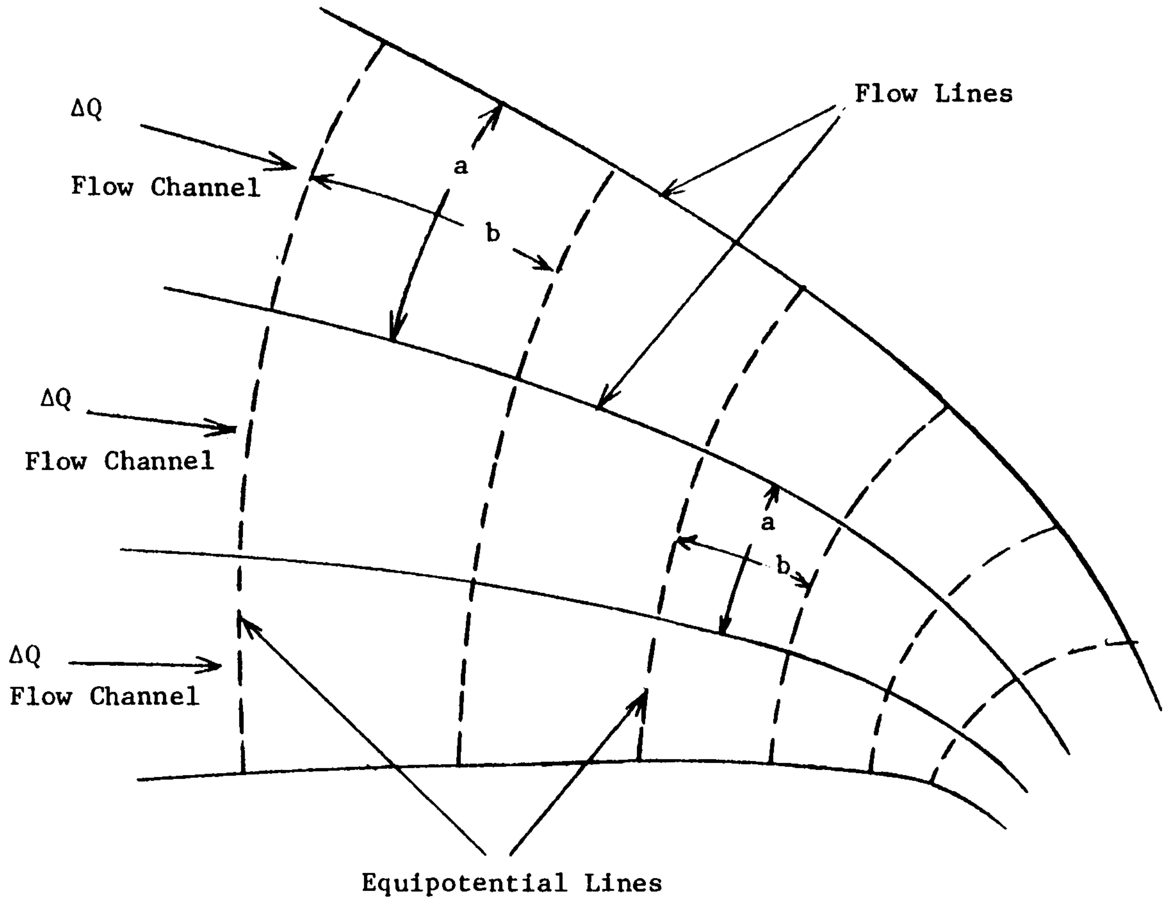


Fig. 2.3 General situation for line of seepage (flow net).

2. The head loss between any two equipotential lines is the same throughout the flow net.
3. The velocity of flow and the hydraulic gradient are inversely proportional to the spacing between the equipotential lines.
4. The seepage forces are maximum at points where flow lines or equipotential lines narrow.

These important general features will be discussed in more detail later. The solution of a properly posed seepage problem constituted of Laplace's equation and boundary conditions requires a solution scheme. Techniques for solving steady state boundary value field problems are discussed in Section 2.5.

2.4.2 Boundary Conditions in Two Dimensional Steady Flow

When dealing with steady state flow through soils, we encounter four kinds of boundaries in the flow region, as shown in Fig. 2.4:

1. Impervious boundary (A-E); where the flow cannot take place across the boundary. As a result, this boundary is also a flow line.
2. Surface of water reservoir (A-B); where the water pressure is distributed according to laws of hydrostatics.
3. Surface of seepage (C-D); where the seepage leaving the porous flow region enters a free surface exposed to the atmosphere. This boundary is not readily defined.

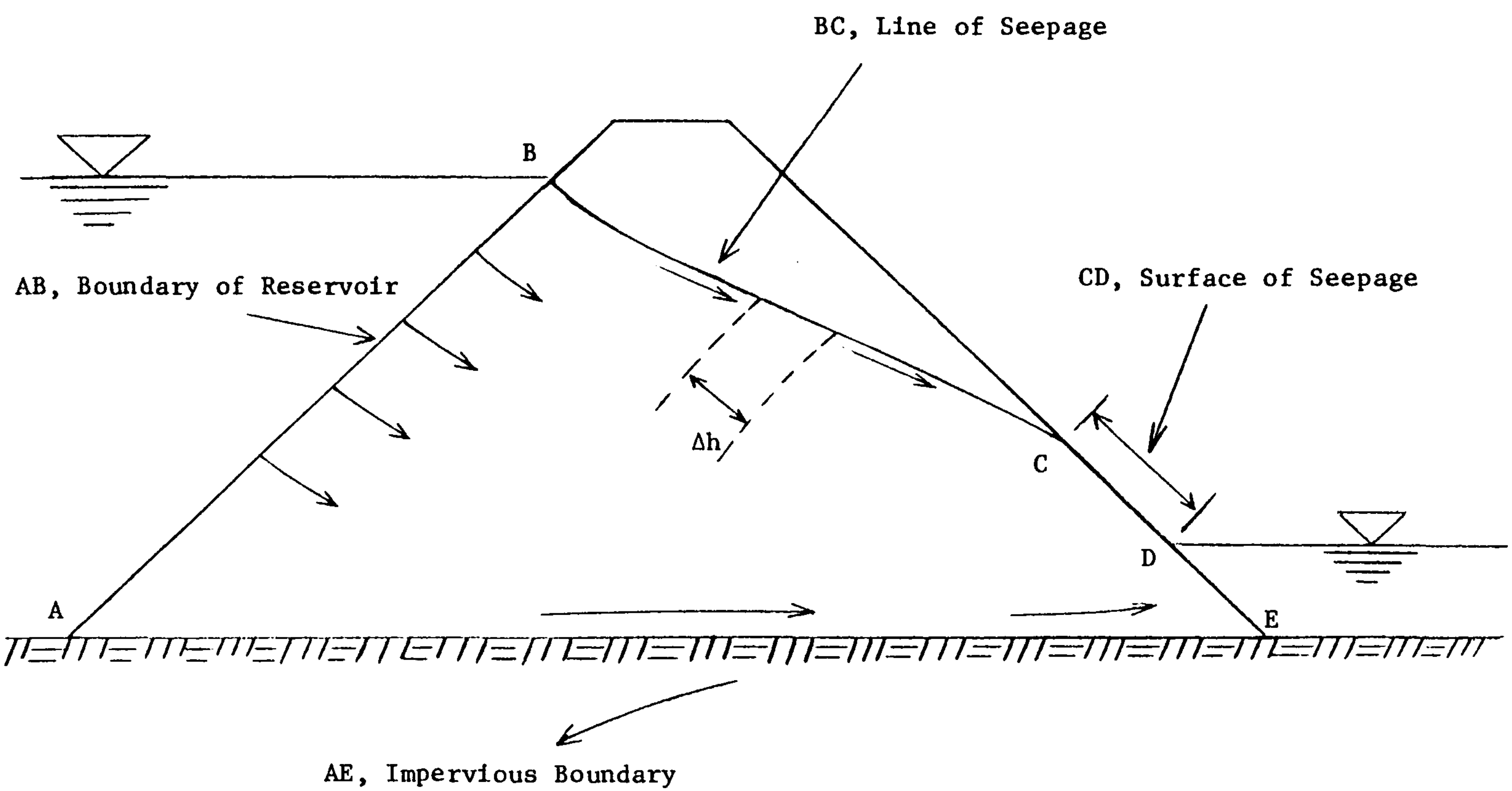


Fig. 2.4 Dam cross section (boundary conditions).

4. Line of seepage (Phreatic Line) (B-C); where the upper stream line (flow line) is a free water surface and has a pressure head equal to zero. The velocity is also equal to zero across this surface as with all flow lines. This boundary is defined by principles and laws discussed later.

The entrance of emergence conditions for the line of seepage through cross sections in which the discharge slope forms an angle with the horizontal between 0° and 180° are not easily described, but numerous analytical and experimental solutions exist [Ref. 1, 10]. An illustration of these are shown in Fig. 2.5.

2.5 Flow Nets and Other Solutions of Steady State Flow Problems in Soils

The properties of the flow net have been described in the previous section. It provides for a representation of the direction of flow and the hydraulic head existing at any point in the soil material through which water is flowing. The solution of seepage problems is usually very complicated to obtain, especially for flow regions of complex geometry and various material properties.

The most effortless method for obtaining the flow net is the Forchheimer sketching scheme, and it is probably the simplest solution procedure available. It's disadvantage is that considerable experience is usually required for an accurate solution. Once sufficient skill is obtained, it presents many benefits to the solution of most seepage problems. These features will be discussed in the following pages.

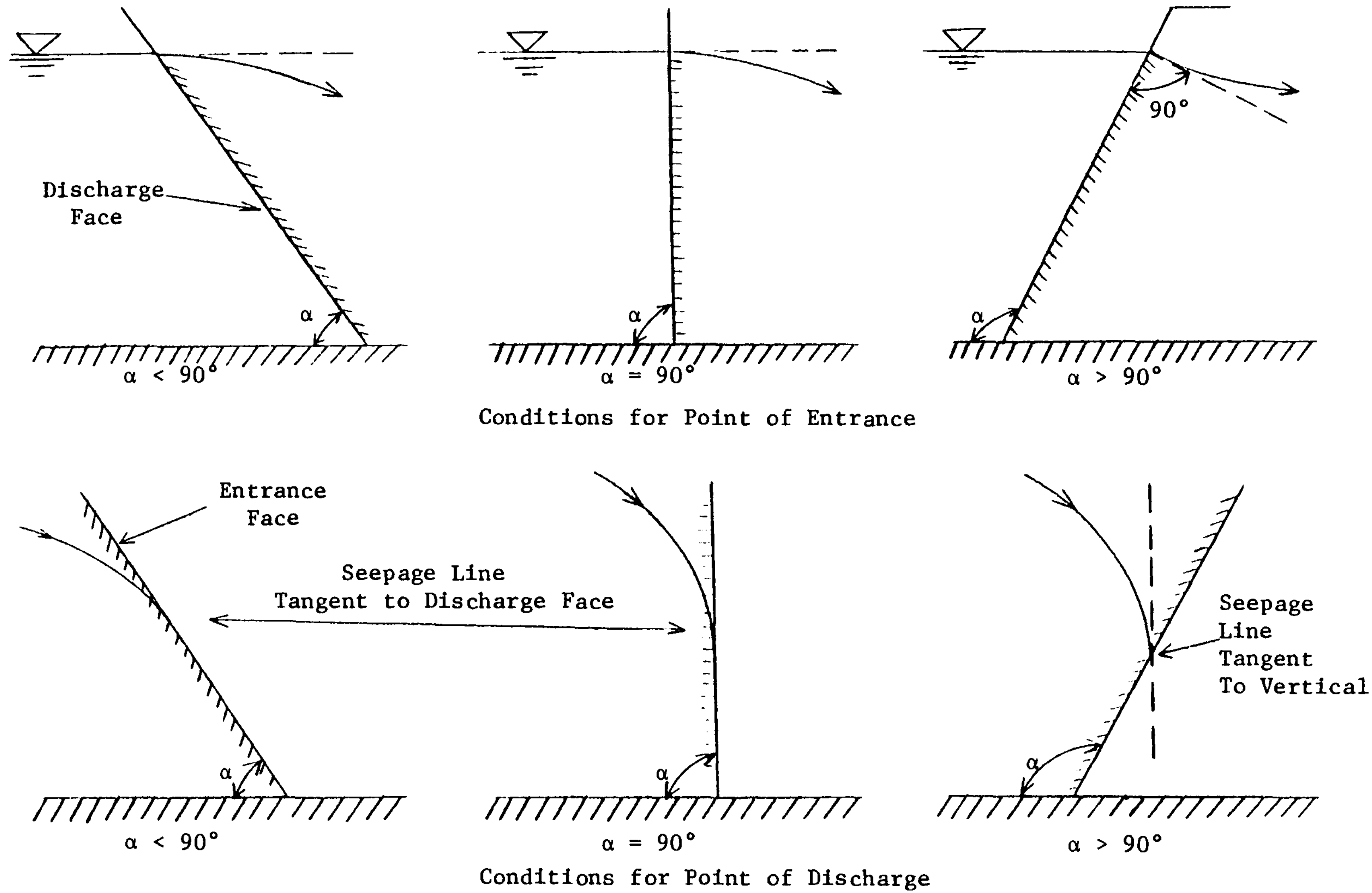


Fig. 2.5 Entrance discharge and transfer conditions of seepage line.

The graphical solutions are obtained by drawing flow lines and equipotential lines in such a way that they intersect to form approximate curvilinear quadrilaterals, later termed "squares." The equipotential and flow lines intersect at right angles for homogeneous and isotropic material, as shown in Fig. 2.3. The closer the lines are spaced, the greater is the accuracy of the solution. This procedure is applicable to any problem of steady state flow in two dimensions and can be used to determine the rate of flow, the head at any point, and the gradient at any point.

After having plotted a flow net that satisfies the necessary conditions, one can compute the desired information. These features will be discussed subsequently:

1. Quantity of seepage (Q) can be derived from Darcy's Law.

   Referring to Fig. 2.3, the following expression may be written:

$$\Delta Q = K\ i\ A$$

$$\Delta Q = k\frac{\Delta h}{b} \cdot a$$

where

ΔQ is the flow channel between ay pair of flow lines

Δh is the head loss between any pair of equipotential lines (equipotential drop) $(\Delta h = \frac{h_L}{N_d})$

a is the width of any flow net element

b is the distance between equipotential lines in such an element

h_L is the total head loss

N_d is the number of squares between two neighboring flow lines.

Rearranging the above expression, we obtain

$$\Delta Q = K \frac{h_L}{N_d} \cdot \frac{a}{b}$$

$$Q = \Delta Q \cdot N_f$$

$$Q = N_f \cdot K \frac{h_L}{N_d} \cdot \frac{a}{b}$$

where

N_f is the number of squares between two neighboring equipotential lines.

If N_f and N_d are selected in such a way that $a = b$, then the quantity of flow (Q) is expressed by,

$$Q = K\, h_L \cdot \frac{N_f}{N_d} \tag{2.9}$$

2. The maximum hydraulic gradient (i_{max}) on the discharge surface, which describes the safety against piping [see Chapter 3] can be determined by equation,

$$i_{max} = \frac{\Delta h}{b_{min}} \tag{2.10}$$

where

b_{min} is the smallest distance between the equipotential lines.

Maximum hydraulic gradients are at points in the flow net where the squares are smallest.

3. The seepage forces (F) acting on a segment of unit thickness is written as:

$$F = i\ V\ \gamma_w = i\ \gamma_w \cdot a\ b\ c$$

$$i = \frac{\Delta h}{b}$$

$$F = \Delta h \cdot \gamma_w \cdot a\ c \qquad (2.12)$$

where

A is the area of the segment (A = a b)

a is the width of any flow net element

b is the distance between the equipotential lines

γ_w is the unit weight of water

c is a unit thickness

Usually the percolation of water through a porous material produces seepage forces (F) as a result of the viscous friction between water and the soil pores, and this results in head loss (h_L). The force corresponding to this energy transfer is known as the seepage force. A complete discussion of seepage forces is given in several text books [Ref. 10]. The seepage forces obtained from flow nets can be combined with other forces such as gravitational and inertial forces for stability analyses [see Chapter 4].

Therefore with the aid of a flow net, it is possible to determine critical points in sections where forces due to seeping water is maximum. The computation of exit gradients assists in the design of filters and other drainage facilities. This is discussed in Chapter 3.

Graphical schemes have also been used for flow of water through dam embankments and foundations with different coefficients of permeability (nonhomogeneous soil) between the zones. The boundary between zones of various soils is shown in Fig. 2.6 [Ref. 26]. The flow lines and equipotential lines are both deflected. In other words, the direction of flow at the boundary between soils with different permeabilities and geometries changes. Various types of flow nets for different values of permeability in dams and foundations are shown in Fig. 2.7 [Ref. 7].

In the case of anisotropic soils in which the coefficient of permeability is different in all directions, the governing seepage equation [$K_x \frac{\partial^2 h}{\partial x^2} + K_y \frac{\partial^2 h}{\partial y^2} = 0$] does not provide for a flow and potential field where normal intersection between equipotential lines and flow lines exist. However, the problem can be converted to the simple Laplace equation, given in equation (2.8), through the application of an appropriate geometric transformation of the permeability and spatial direction to produce an ordinary flow net. For example, knowing that the permeability in the horizontal direction is usually greater than the vertical direction, a transformed section can be used, whose horizontal dimension is reduced to correspond to the greater ease of percolation in that direction. This reduction (transformation) can be achieved by multiplying the vlaue of x by the radical

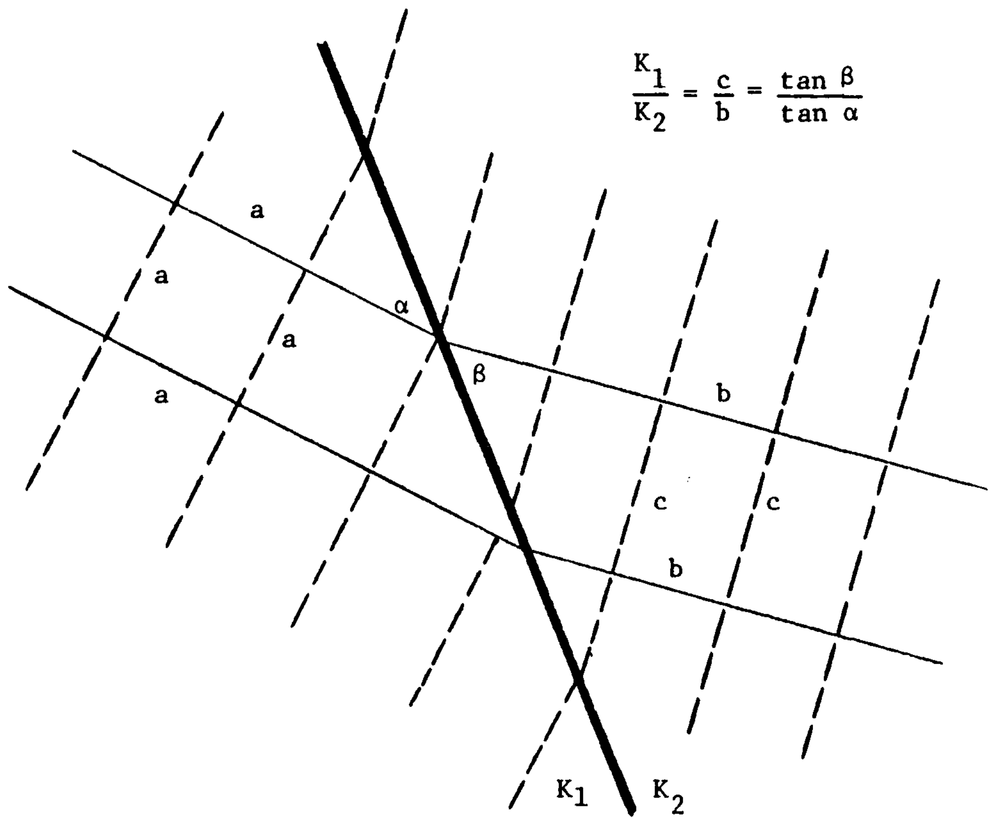


Fig. 2.6 Deflection of flow net at boundary of soils of different permeabilty (K_1, K_2) [Ref. 26].

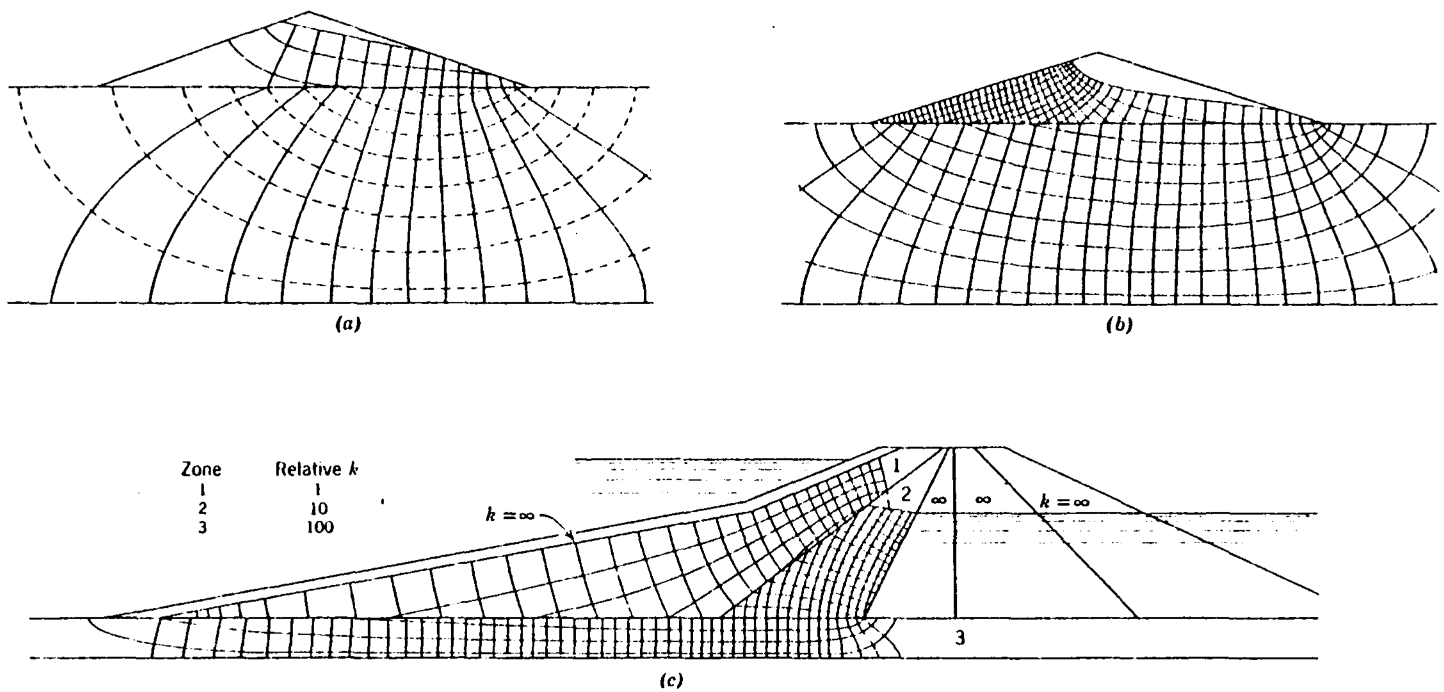


Fig. 2.7 Typical flow nets through dams on foundations of different permeability [Ref. 7]. (a) Permeability of embankment 10 times the permeability of foundation. (b) Permeability of embankment 0.1 times the permeability of foundation.

fraction $(\sqrt{\frac{K_y}{K_x}})$ to produce the new transformed coordinate X as,

$$X = x \left(\frac{K_y}{K_x}\right)^{1/2} \tag{2.13}$$

where

X = transformed horizontal direction (x)

x = real horizontal dimension

Ky = coefficient of permeability in vertical direction

K_x = coefficient of permeability in horizontal direction.

Thus, the general seepage equation becomes,

$$\frac{\partial^2 h}{\left(\frac{K_y}{K_x}\right) \partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

$$\frac{\partial^2 h}{\partial X^2} + \frac{\partial^2 h}{\partial y^2} = 0 \tag{2.14}$$

Equation (2.14) indicates that by reducing the horizontal direction, a given homogeneous and anisotropic flow region can be transformed into a fictitious isotropic region where Laplace's equation is valid. In other words, the flow net in the transformed section has the same characteristics (flow lines normal to equipotential lines). Once the solution is found on the transformed section, the x coordinate is expanded to the true section. A typical transformed solution is shown in Fig. 2.8 [Ref. 7]. Notice that the sketching scheme becomes very complicated where three or more boundary

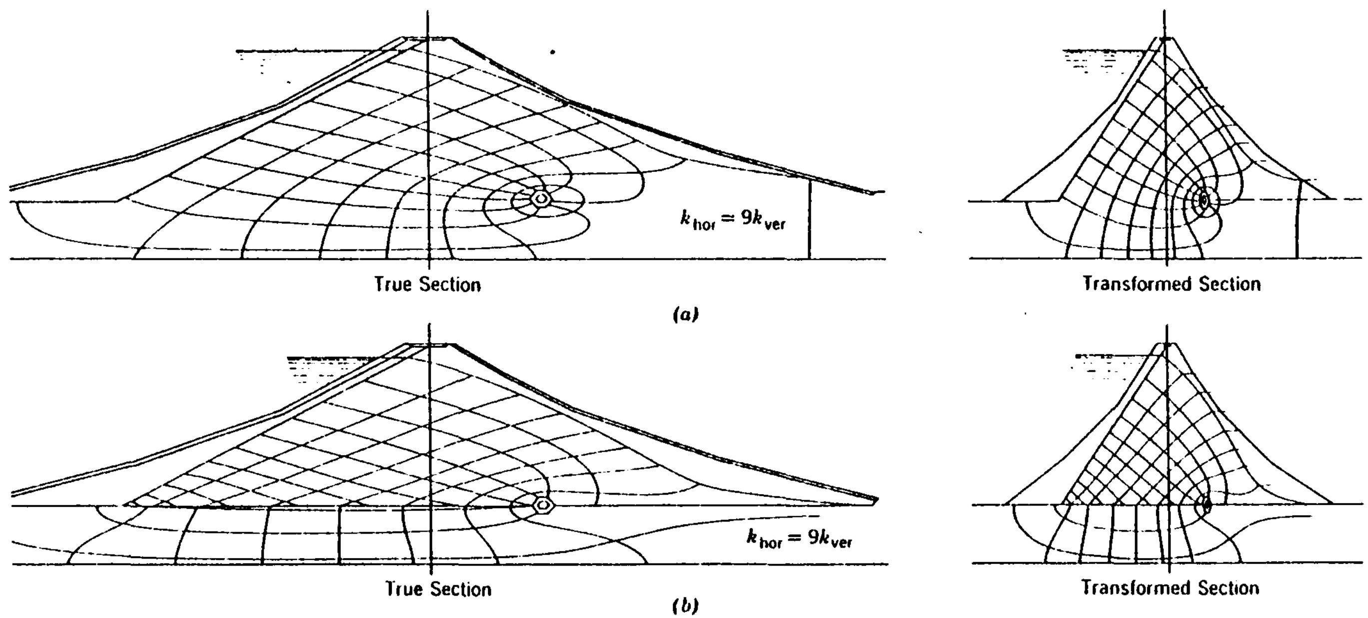


Fig. 2.8 Typical flow nets through anisotropic embankment and foundations [Ref. 7]. (a) Foundation permeability equal to embankment permeability. (b) Foundation permeability 30 times the embankment permeability.

potentials exist. The sketching solution may be augmented by analog field models.

The equivalent coefficient of permeability (K_e) for a homogeneous and anistropic section is given by:

$$K_e = \sqrt{K_x K_y} \tag{2.15}$$

The quantity of seepage for such conditions can be determined by:

$$Q = K_e \, h_L \, (\frac{N_f}{N_d}). \tag{2.16}$$

The percolation of water through the dam will carry soil particles and eventually this water movement will erode the downstream face of the dam, and in severe cases it can lead to total failure of the structure. Dam failures are likely to result in great loss of life and great property damage, because of sudden release of large volumes of water. A complete discussion of seepage effects on earth structures is given in Chapters 3 and 4.

CHAPTER 3

SEEPAGE CONTROL

3.1 General

Earth structures require some form of seepage control system, in order to remain stable and for protection against damaging effects of erosion. Seepage uplift and loss of structural support through erosion may affect the long term stability of a structure if uncontrolled. This may eventually result in the total failure of the earth structure.

3.2 Some Typical Problems Due to Seepage

There are generally three damaging mechanisms related to seepage through the foundation of a dam or through the dam itself.

1. Piping: The percolation of water through a dam, tends to carry soil particles through unprotected exits and boundary surfaces. If the forces resisting erosion are less than those which tend to cause it, the soil particles will be washed away and piping occurs. The movement or washing out of soil particles leads to the development of channels which in turn facilitates further flow. The fluid gradient increases and the erosion is accelerated.
2. Swelling: Swelling occurs in sensitive soils and many soils containing montmorillonite clays. Swelling soils should be

avoided in dam construction. They can, however, occur in or near the foundation regime. The general swelling mechanism results in soil expansion when the confining pressure is reduced or the water content of the soil is increased.

3. Excessive Loss of Water: Large amounts of water are often lost through the breaching of dams, or overtopping in conjunction with sudden heavy rainfalls or earthquake action.

Filters, impervious blankets, cutoff walls and trenches are among the various methods for controlling seepage. These will be discussed in the next section.

3.3 Methods Used for Controlling Seepage

Before any type of control is used, the foundation should be thoroughly explored to assure that the feature chosen will apply to the conditions locally met and will serve the purpose proposed. In many situations, a combination of different schemes can be used. This is especially true when the seepage through the embankment and foundation (under seepage) are associated.

Horizontal drainage blankets, vertical drains and toe drains provide seepage control throughout the dam cross section but especially in the downstream segment of the dam. A combination of inclined and horizontal filters are often used for reducing the internal seepage. Seepage through the dam can also be minimized by placing a segment of less pervious soil near the center profile or on the upstream face of the dam. Some of these methods and features will be discussed in the following sections.

3.3.1 Filter Criteria

Filters permit the water to escape freely, but they keep the soil particles in place. Filters are installed near seepage discharge regions, which may be subject to piping or surface erosion.

Cemented soils and sound rocks which are not susceptible to erosion, do not require filters, but residual soils, weathered rocks, friable sandstones, and chemically soluable rocks such as lime stone must be protected by a filter structure [Ref. 1, 4]. In designing filters for drains, the filter material should be more pervious than the fine base material. This permits free passage of water and reduces the head.

The following criteria have been experimentally established by Terzaghi [Ref. 7], and they were verified experimentally by Bertram (1940) [Ref. 4], who also established that the criteria satisfy general filter stability standards. An adequate increase in permeability between base and filter is provided by these criteria. They apply to nearly all filter geometries. These criteria are satisfactory for natural sands, gravels and crushed rock.

$$\frac{D_{15}\text{ (of the filter)}}{D_{85}\text{ (of the soil)}} < 4 \text{ or } 5 \tag{3.1}$$

$$\frac{D_{15}\text{ (of the filter)}}{D_{15}\text{ (of the soil)}} > 4 \text{ or } 5 \tag{3.2}$$

where

D_{15} = the diameter at which 15 percent of the total soil particle mass are smaller

D_{85} = the diameter at which 85 percent of the total soil particle mass are smaller

The above criteria have been discussed by many investigators, and they have been substantiated by field observations. Most practitioners agree that when the basic criterion reflected in equation (3.1) is fulfilled in every part of a graded filter, piping will not occur under even large hydraulic gradients. The criteria appear to be very conservative and adequate for any soil type [Ref. 4]. They have been found to be overly conservative for clays and coarse soils with clayey fines, which have inherent resistance to piping because of their cohesion.

In addition to the limiting criteria discussed earlier, a maximum particle size of 3 inches should be used in order to minimze segregation and bridging of large particles during placement of filter materials. The material is usually spread in layers during placement and the spreading equipment often tends to separate the coarser gravels and move them to the side. Sherard et al. (1963) [Ref. 7] recommend a minimum 6 inch thickness of sand for horizontal layers of filter and 12 inch for gravel. For vertical or inclined filters, a minimum horizontal width of 8 to 10 ft has been recommended.

3.3.2 General Drainage Method and Design Criteria

Any earth dam which is composed of nearly impervious or semi-pervious soils, should be provided with some type of downstream drain. Drains are used for the following purposes;

1. to control and collect seepage at the downstream portion of the dam
2. to reduce the pore water pressure in the downstream portion of the dam which in turn increases the stability of the downstream embankment
3. to ensure that important soil characteristics such as dry unit weight, moisture content, and soil fabric are maintained.

The effectiveness of the drain in reducing pore water pressure depends on its location and extent. Filters and general drainage systems serve the same purpose in regions where piping or liquefaction may occur, and they should, in that case, be designed according to the criteria given in equations (3.1, 3.2).

The design features of drains depend on the drainage characteristics of the soils, the operational life span of the system, environmental or seasonal effects, and they also depend on the position of the ground water table. In order to design a drain it is necessary to estimate the minimum permeability and thickness of the drain. This can be achieved by flow nets or by Darcy's Law as discussed earlier. Flow nets can be constructed in order to compute, by a trial and error scheme, the required dimensions to ensure adequate discharge capacity of the drains. An illustration of such a flow net is shown in Fig. 3.1 [Ref. 4]. The total seepage rate through the drain can be calculated from flow net construction in which it is assumed that the drains have an infinite permeability [Ref. 4]. In the case of a horizontal drain, the designer must make sure that the line of seepage does not rise to the top of the drain. Knowing the permeability of the material, K_3, the thickness of the drain h_3 can be computed based on an expression developed by Dupuit (1863),

$$Q = \frac{K_3 \, h^2{}_3}{2L_3} \tag{3.3}$$

Dupuit assumed that for small inclinations of the line of seepage the streamlines can be taken as horizontal, and the hydraulic gradient is equal to the slope of free surface and is invariant with depth. A complete discussion of Dupuit's assumptions is given in various text books [Ref. 10].

The following drains are used in homogeneous dams:

a) Horizontal Drainage Blanket

Drainage blankets are used to control the seepage from both the foundation and the embankment, in those instances where the structure is built on exposed sands and gravels. Such a drain should be placed properly over the downstream abutment to collect seepage emerging from the abutment and to direct the flow to the controlling

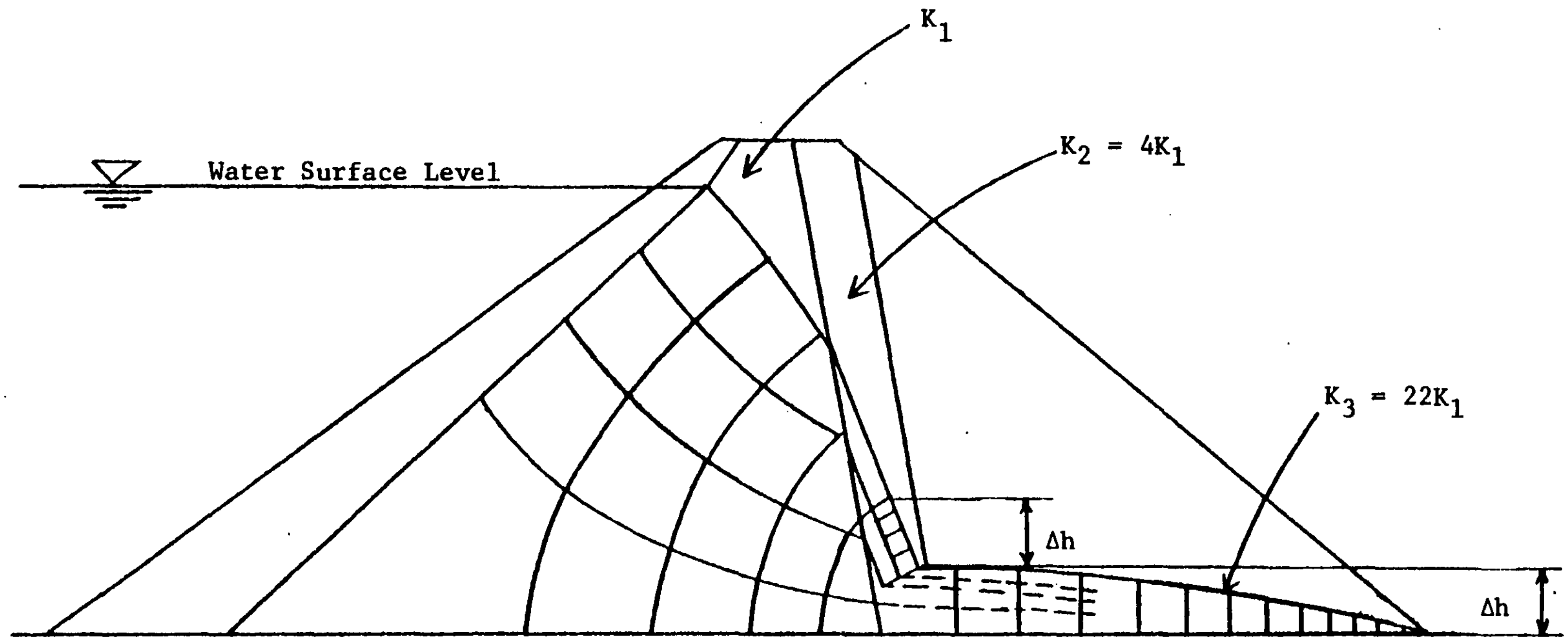


Fig. 3.1 Flow net for core and drains [Ref. 4].

outlet [see Fig. 3.2]. The important function of a horizontal blanket is to lower the phreatic line so as to prevent erosion and freezing in the downstream segment of the dam.

b) Toe Drains

Toe drains are often installed along the downstream toe of earth dams in conjunction with a horizontal drainage blanket. The purpose of the drain is to collect seepage from the foundation, and the embankment. The discharge from the drain is usually collected by pipes (Fig. 3.3a) which lead into the river channel below the dam. The drain pipes are placed in the trenches at an adequate depth below the ground surface to insure effective interception of the seepage.

Typical toe drain installation and design details are shown in Fig. 3.3.b. Toe drains are installed in the bottom of the trench, and material satisfying the filter criteria, discussed in Section 3.3.1, is used as backfill.

3.3.3 Seepage Reduction

Seepage reduction can be achieved by using cutoffs such as concrete walls, steel piling, impervious cores, grout curtains, impervious membranes on the upstream face, upstream impervious soil blankets, slurry trenches, and relief wells. The results obtained by these methods show that the water pressure and seepage forces are reduced in critical exit regions [Ref. 4].

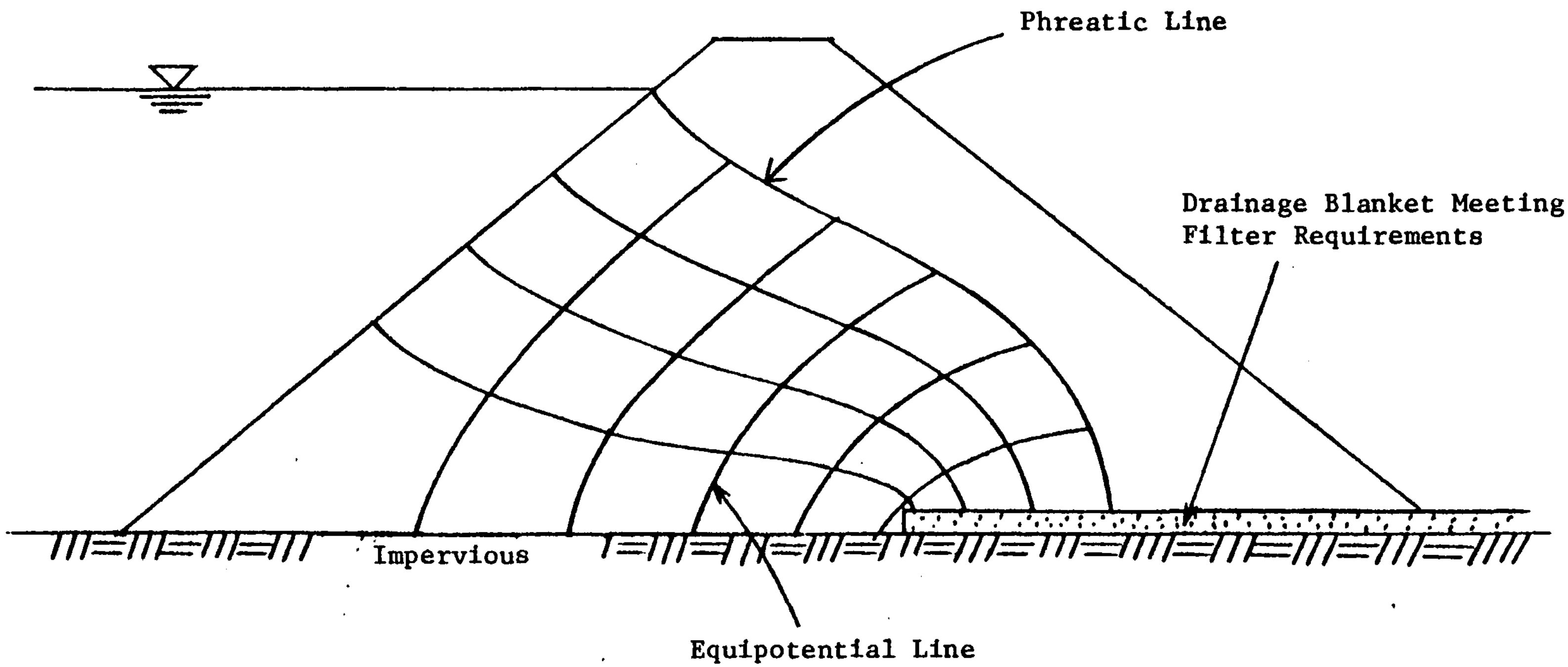


Fig. 3.2 Flow net with horizontal drainage blanket.

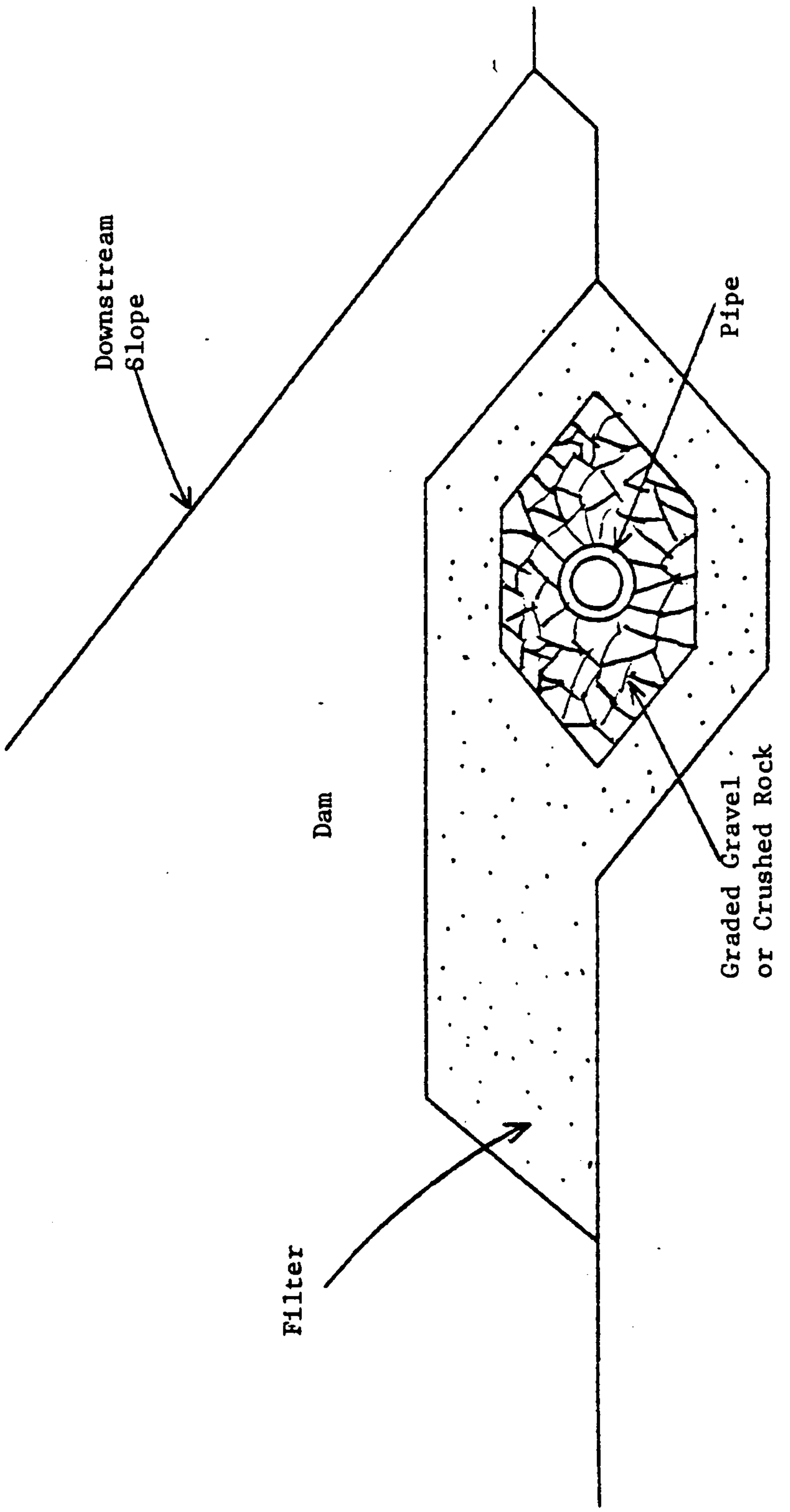


Fig. 3.3a Typical toe drain.

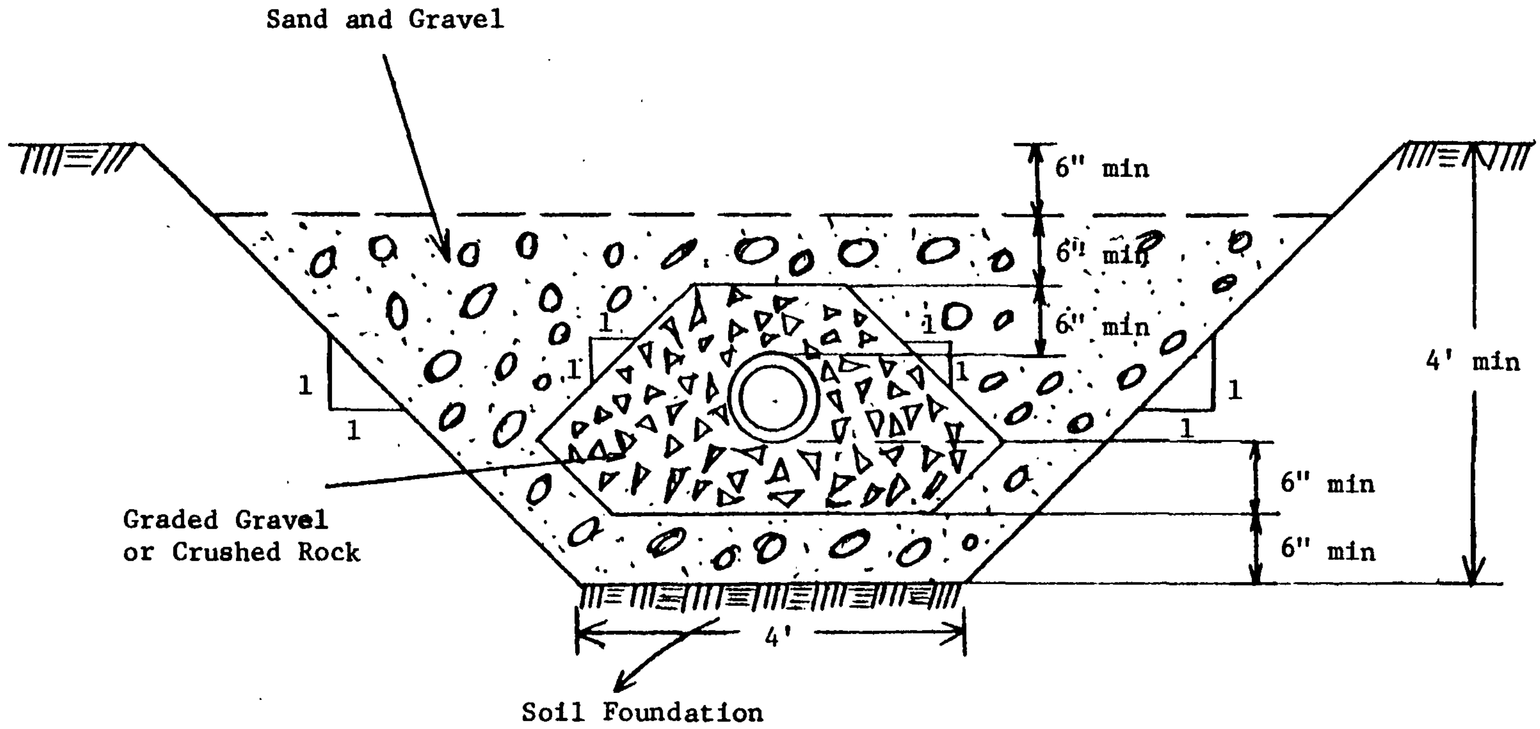


Fig. 3.3b Typical toe drain installation [Ref. 10].

Some of the design features for controlling seepage in earth dams are illustrated in Fig. 3.4. These methods are often used in combination with properly designed filter and drainage structures.

The following features are used to reduce seepage:

a) Cutoff Trenches

Cutoff trenches are used to reduce the percolation of water under and outside the dams and to control the seepage forces. Reduction of seepage can be obtained through the use of compacted backfill trenches. The cutoff trench is an extension of the impervious zone. The base width of the trench is about a quarter of the net difference between the maximum upstream water level and minimum downstream tail water elevation [Ref. 24]. The backfill material used for trenches are from well graded mixtures of gravel, sand, silt or clays which produce a flexible material. Great care must be applied to make sure that the base of the trench is located on bedrock and not on large boulders. The location of the trench is usually upstream from the center line of the dam as shown in Fig. 3.5 [Ref. 7].

Turnbull and Creager [Ref. 22] conducted experiments on homogeneous isotropic pervious foundations, and they concluded that a cutoff trench extending 50 percent of the distance to the impervious stratum will reduce the seepage by only 25 percent, and that a 80 percent cutoff penetration may be required to reduce the seepage 50 percent.

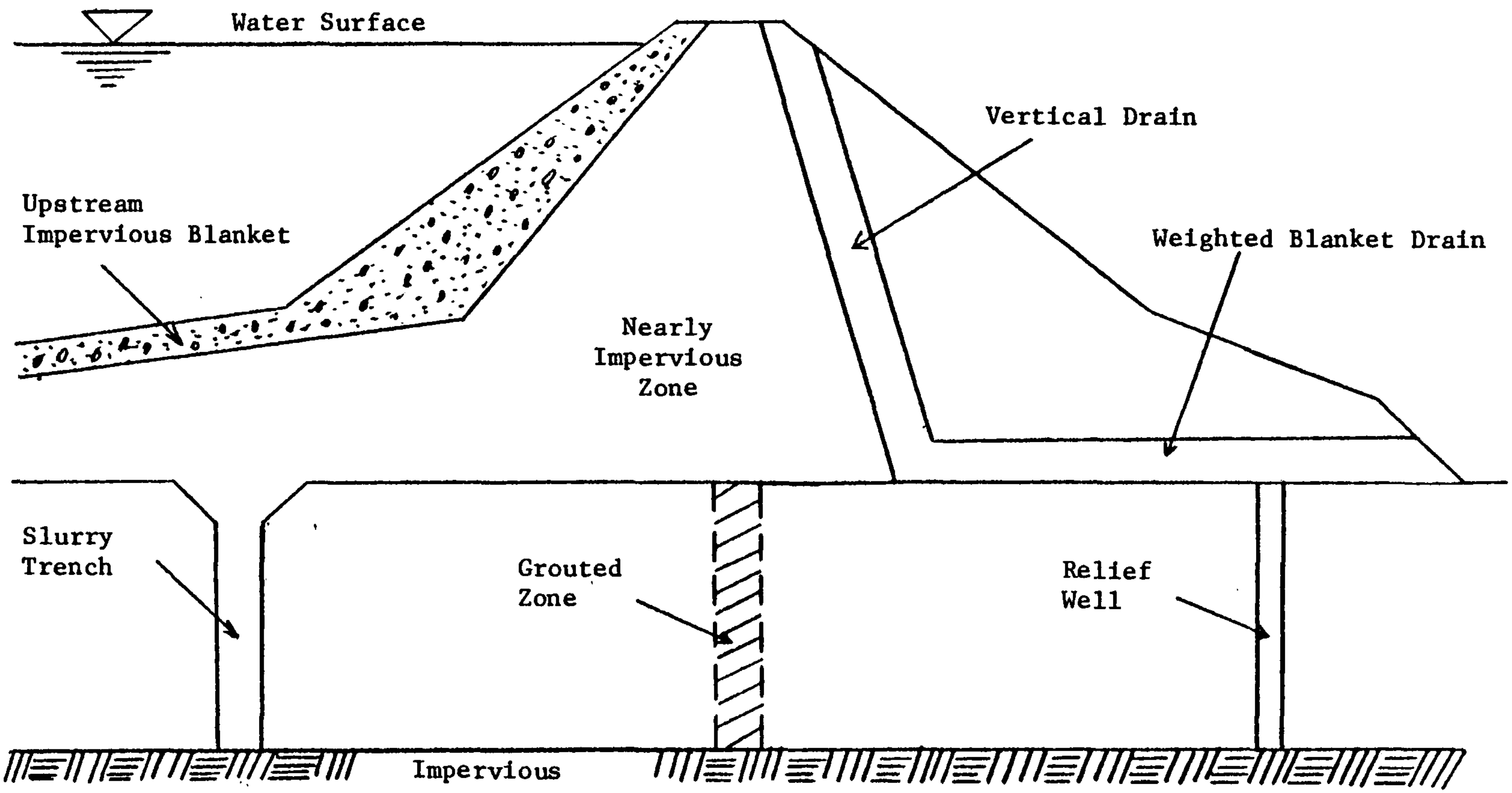


Fig. 3.4 Design features for controlling seepage in earth dams.

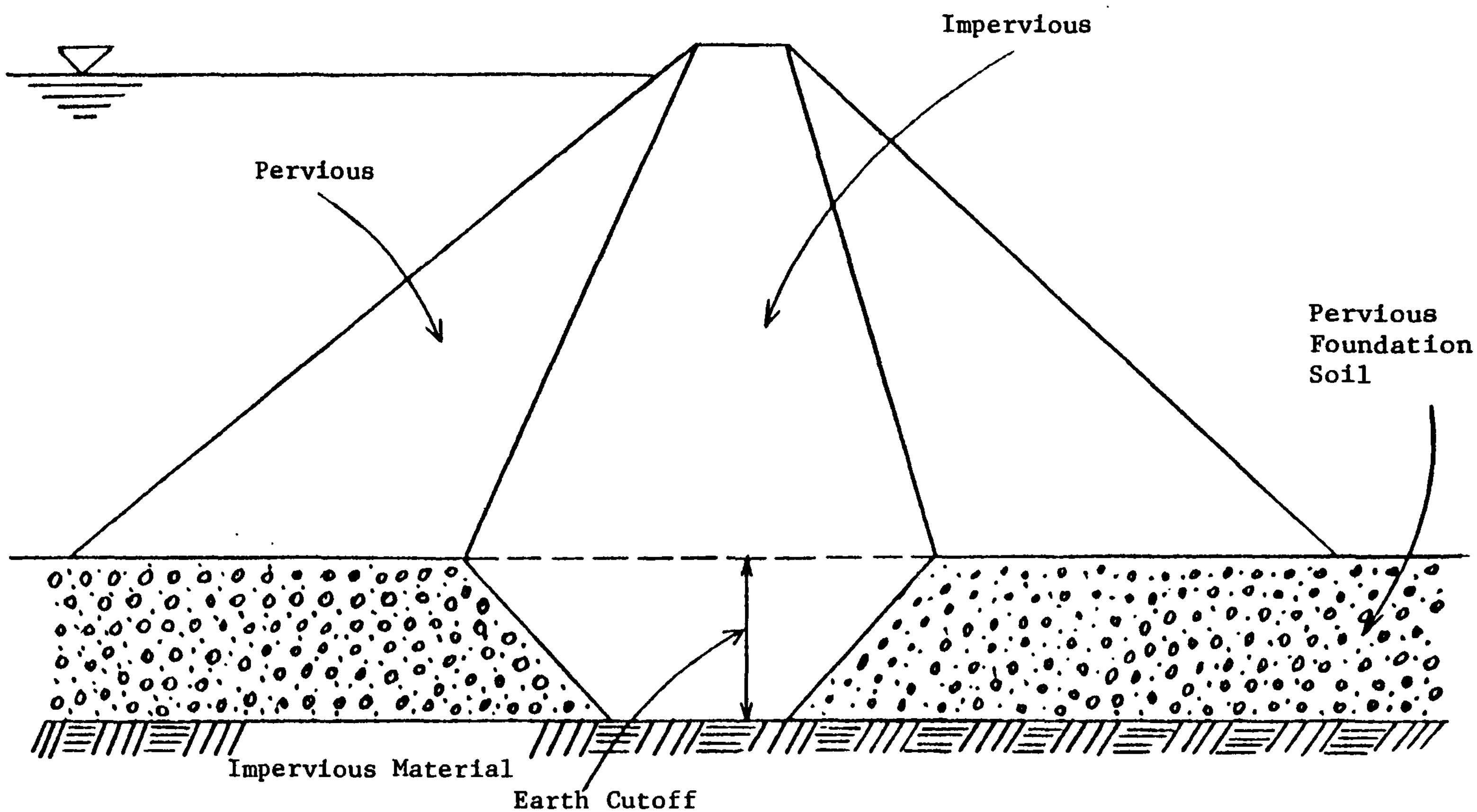


Fig. 3.5 Typical foundation cutoff trench.

In order to give a more complete discussion of this situation, we will consider the following example.

Fig. 3.6.a-c illustrate the effect of partial cutoff on the geometry of flow nets. Suppose that the dam shown in Fig. 3.6.a was constructed on a foundation with a coefficient of permeability equal to that of the downstream (pervious) zone. The line of seepage could rise to a high level, and large exit gradients might exist at the toe.

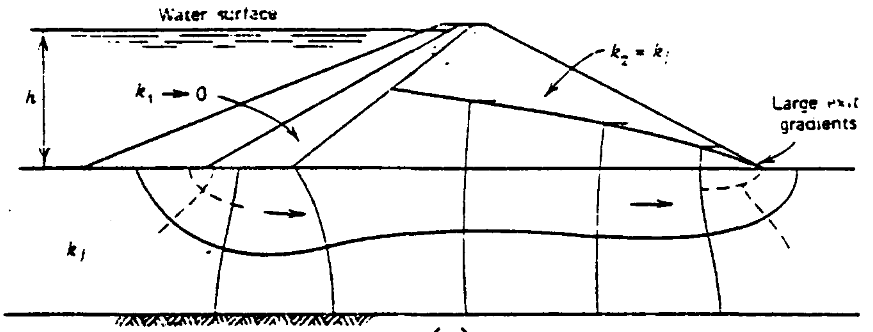
The following methods are recommended by Casagrande for lowering the height of the line of seepage and exit gradients [Ref. 4].

1. The downstream shell should be constructed 100 times more permeable than the foundation.
2. Impervious cutoff trenches should be installed into the foundation.
3. The foundation beneath the core should be grouted.
4. An impervious blanket upstream from the core should be installed.

These guidelines will reduce the seepage of water through the dam.

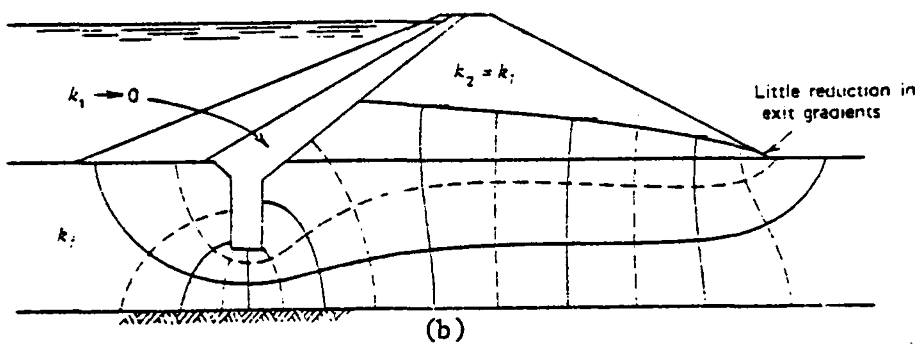
In Fig. 3.6.b, an impervious cutoff is installed to depth equal to 60 percent of the depth of the pervious foundation. The flow net construction shows that water rises slightly lower than that shown in Fig. 3.6.a, and the exit gradients are reduced slightly.

Fig. 3.6.c shows the position of the line of seepage for different depths of cutoff trenches. The cutoff has to be designed



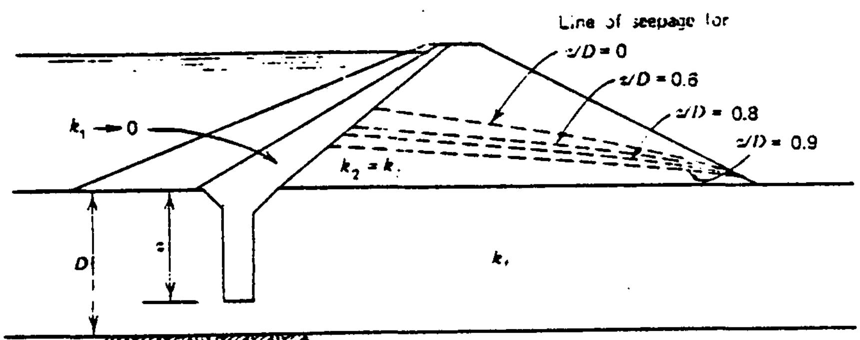
(a)

Flow Net ($\frac{Z}{D}$ = 0)



(b)

Flow Net ($\frac{Z}{D}$ = 0.6)



(c)

Position of Line of Seepage for Various Values of $\frac{Z}{D}$

Fig. 3.6 Response of partial cutoff on the line of seepage [Ref. 4].

properly, in order to have influence on the height of the line of seepage. This situation is very difficult to achieve, and therefore it is often required to use other seepage control schemes in addition to cutoff trenches [Ref. 4]. Such techniques frequently involve sheet piling.

b) Sheet Piling Cutoffs

A sheet piling cutoff is a seepage barrier used in pervious soils to reduce seepage especially in those instances where earth cutoffs are impractical. The sheet pile cutoffs were used more frequently in the past in conjunction with earth dams, but their use has diminished in recent years. They are expensive and experience has shown that leakage through the interlocks between individual piles can be considerable [Ref. 7]. In spite of these problems they are often used in those circumstances where conventional cutoff methods are too expensive or infeasible. They are also used in combination with partial cutoff trenches as a means for increasing the depth of the cutoff.

Before driving sheet piles, it is required to seal the interlocks by injecting bentonitic mud or another form of resin (Sherard, 1968). This is done because leakage can occur through the interlocks or through locations where the sheets are driven out of the interlocks. Good contact is needed between the bottom of the piling and the impervious stratum. If the sheet piling is placed properly, one can expect the seepage to be reduced 80 percent or more [Ref. 9]. If the pile is not placed on impervious stratum, then we can expect 50 percent of the seepage to be effective.

c) Foundation Grouting

The seepage through the foundation must be controlled so that no internal erosion takes place and in order to prevent sloughing in the area where the seepage emerges. The necessary requirements of a foundation for an earth dam are that the foundation should provide stable support for the embankment under the conditions of saturation and loading and that it provide sufficient resistance to seepage.

To satisfy these conditions, it is necessary to evaluate whether or not the foundation should be grouted. This can be accomplished by examining the site geology and by performing in-situ permeability water loss tests by means of injecting pressurized water into drill holes. The time it takes for the pressure to dissipate or other criteria can be used in the field evaluation.

The purpose of grouting is to reduce seepage through the rock joints and faults. Foundation grouting is a process of injecting a viscous fluid sealing material into the rock under pressure into the openings, for the purpose of sealing off or filling joints, seams, fissures, and other openings.

The grout consists usually of a mixture of cement and water, starting with the ratio of 5:1 [Ref. 7]. Grout mixes vary between 10:1 and 0.8:1. If the grout absorbtion is too high, sand is added to give the grout additional bulk. Bentonite is combined with the sand in small quantities, about 2 percent by weight of the cement, in order to obtain a pumpable grout mix. The U.S. Bureau of Reclamation recommends the gradation of sand and gravel, shown in Table 3.1, as part of grouting mixtures [Ref. 4].

Care must be exercised so that the pressure is not so high as to cause hydro fracturing in the rock, but it should be high enough to thoroughly fill all of the interstices of the rock. The pressure can be increased by delaying the grouting until an overburden of masonry has been placed on the foundation or by providing a suitable means in the grouting pipe for releasing pressure at various depths in the grout hole. The grouting of the foundation is performed along a single line of grout holes, with center to center distances from 10 to 20 feet [Ref. 9]. This creates a broader seepage barrier zone called grout curtain. Multiple lines of grout holes can be used for the rock area just below the soil. In some cases where large zones of fractured rock lie at the foundation contact, the entire zone of fractured rock needs to be grouted to shallow depth, usually 10 to 30 feet. This type of grouting reduces leakage in the fractured zone and provides a more firm foundation for the dam.

d) Upstream Impervious Blankets

The impervious blankets are placed upstream of the dam to increase the path of seepage in a pervious foundation, which in turn, reduces the exit gradients, and the quantity of seepage. Blankets

TABLE 3.1

Gradation of sand and gravel used for grouting

Screen No.	Cumulative percent, by weight retained on screen
8.	0
16.	0 to 5
30.	15 to 40
50.	50 to 80
100.	70 to 90
200.	95 to 100

are used when cutoffs to bedrock or to an impervious layer are not practicable because of excessive depth [Ref. 9]. They are also used in conjunction with partial cutoff trenches. The proper design of length and thickness of a blanket depends on the permeability of the blanket material, the thickness of the pervious foundation, and the depth of the reservoir. The thickness of the blanket is between 2 to 10 ft. The length of the blanket is governed by the desired reduction in the amount of underseepage. As discussed in Chapter 2, the rate of seepage is inversely proportional to the length of the path. A complete discussion of upstream blanket design and analysis procedures is given by Bennett (1946) [Ref. 25].

Many times a natural blanket in the form of clay or fine silt exists, ranging in depth from a few feet to many feet. For such cases, it is beneficial to take full advantage of the natural formation and consequently the exploitation of upstream reservoir borrow should be controlled to prevent excessive excavation of the natural impervious top blanket.

e) Relief Wells

Relief wells are used to control the pore water pressure caused by percolation of water under the foundation of an earth dam. They are frequently used in cases where impervious blankets protect the upstream surface. Relief wells are also used to reduce the quantity of uncontrolled seepage downstream of the dam. A typical relief well is shown in Fig. 3.7.

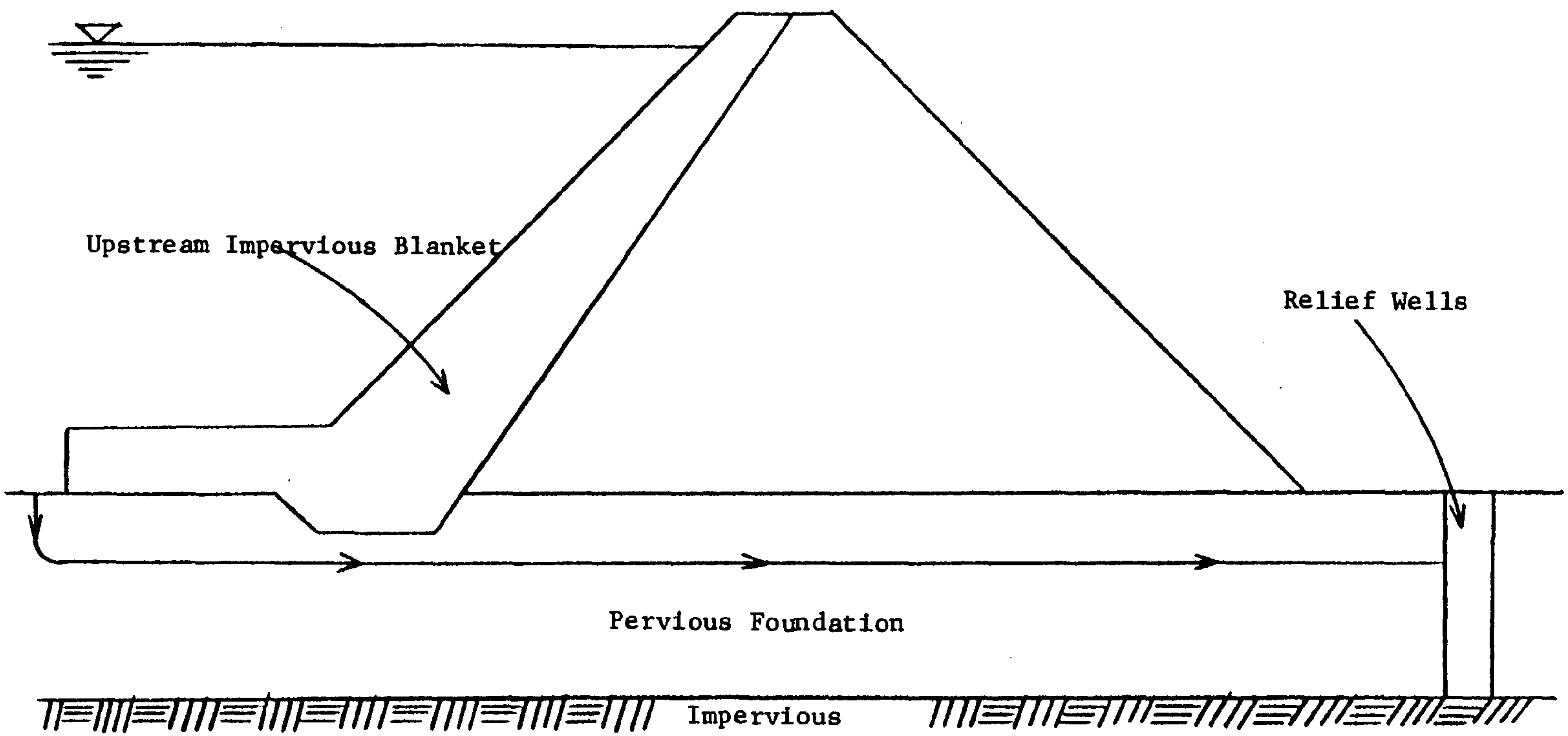


Fig. 3.7 Typical relief wells, used to control seepage under the dam.

Casagrande (1961) pointed out the importance of extending the relief wells deep enough into the foundation soil. This is especially important for soil formation where the seepage pattern cannot be correctly estimated in advance. Some design and installation features regarding relief wells are presented by Sherard et al. (1963) [Ref. 7] and the U.S. Corps of Engineers [Ref. 9].

Relief wells are usually connected to drainage galleries which extend underneath embankments and into abutments as shown in Fig. 3.8 [Ref. 8]. Such a system was constructed at El Infiernillo Dam in Mexico. Galleries are constructed for drainage purposes and for the purpose of continuous visual subsurface structural inspection of the dam and its foundation. They are located in the rock underneath embankments, in transition zones, and in the core of the dam.

Fig. 3.9 shows a cross section of the Aswan Dam in Egypt [Ref. 8]. The dam is equipped with three upstream blankets and in the vertical grout curtains penetrating the full depth of the pervious sediments to the granite bed, and downstream relief well. Aswan Dam is protected by two independent seepage control features [see Fig. 3.9]; an upstream blanket and a vertical grout certain. Relief wells are used to reduce the excessive pressure gradients at the downstream exit faces.

3.4 Placement and Compaction of Core Material

The material used for dam cores generally consist of silts, clays, fine granular soils with some silt, coarse grained soils, such

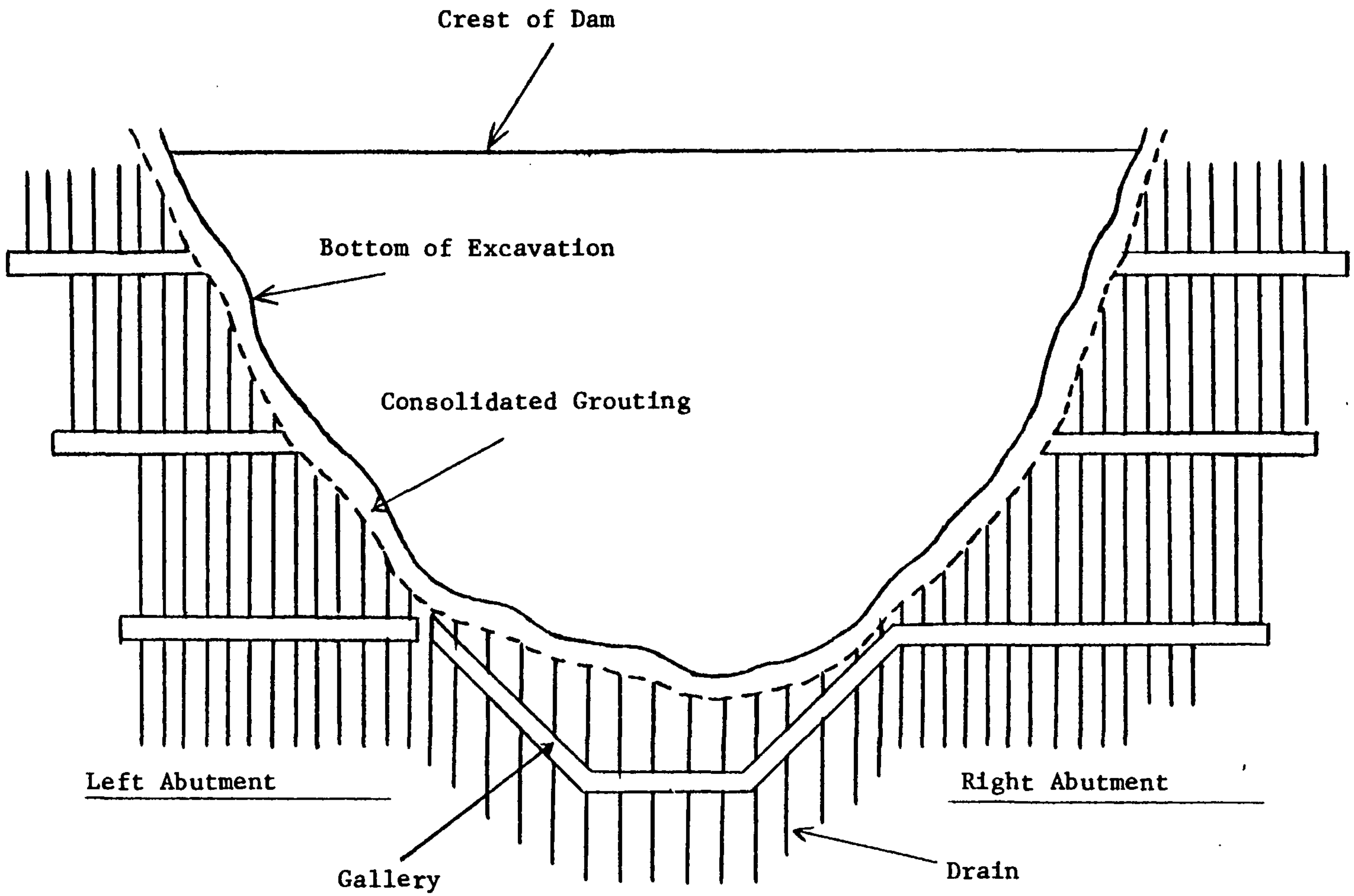


Fig. 3.8 System of drainage galleries at El Infiernillo Dam, Mexico [Ref. 8].

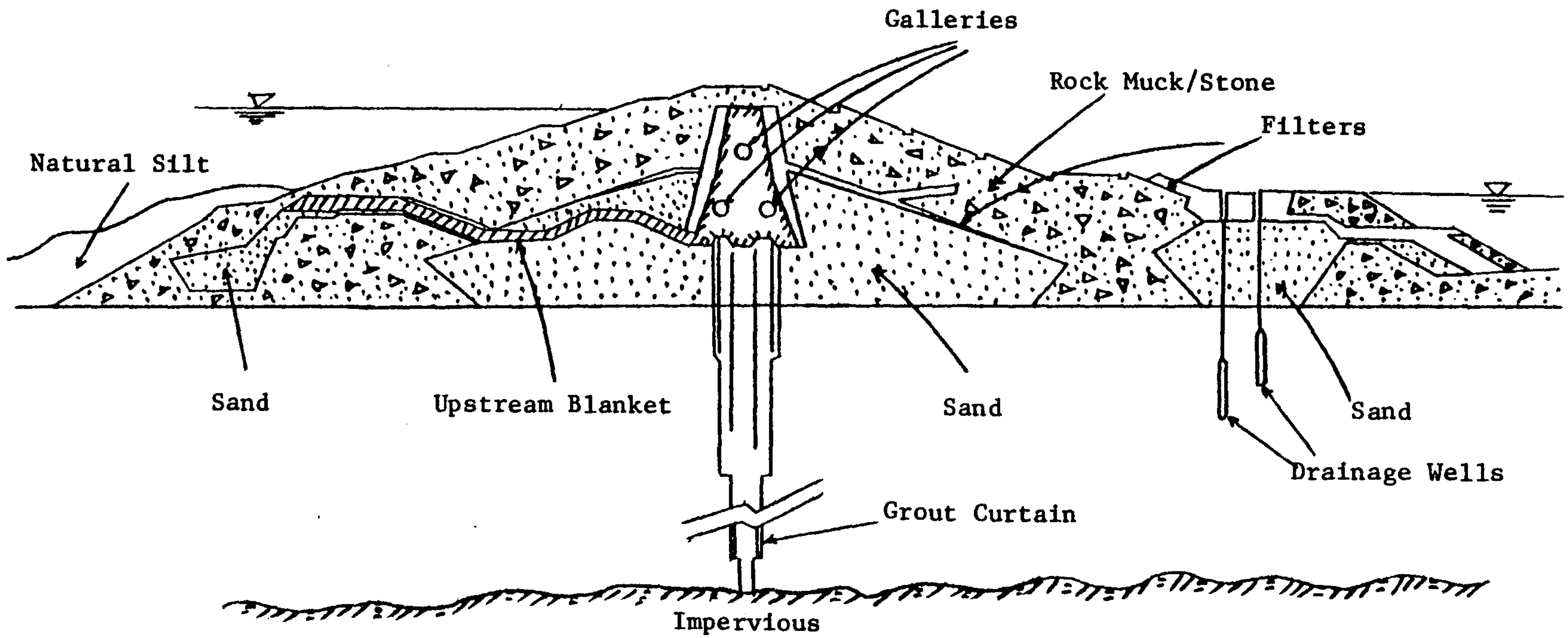


Fig. 3.9 Schematic cross section of High Aswan Dam, Egypt [Ref. 8].

as glacial till or moraines, that contain some percentage of gravel. If there is a choice between different soils, it is required to select the soil offering best resistance to piping. Whatever, regardless of the material used, the thicker the core is the greater will the piping resistance be. In order to reduce the seepage through cores, the moisture content of the core materials needs to be adjusted, either by drying, using admixtures, or by adding water so that strength and permeability and flexibility are achieved in the compaction process. Generally, limiting moisture contents exist for any particular core material; the lower limit is that which will not result in additional settlement of the core upon saturation; the upper limit is that which makes the material difficult to work with, place and compact. These limiting values take into account both economic considerations and design requirements.

Methods used to place and compact the core are different from one dam to another and from one region to another [Ref. 8]. Usually the maximum allowable particle size is specified to range from 1/2 to 2/3 of the maximum lift thickness and the lift thicknesses are adjusted so that placement procedures do not result in segregation, and compaction results in fairly uniform densities with depth.

In practice the cores of most earth dams are placed and compacted on the wet side optimum moisture content. For example, at Netzahualcoyott Dam in Mexico, the core was placed at moisture contents well above optimum; about plus 7 percent [Ref. 7]. The core material consists of silt derived from a weathering of a weakly cemented conglomerate.

Most earth dams constructed by the U.S. Corps of Engineers are compacted with a plastic core section, also placed at an average moisture content slightly wet of optimum. From experience, it is advantageous to place the lower portions of the core at water contents below optimum to reduce pore pressures and compressibility, and to place the upper portions and those zones adjacent to the abutments above optimum in order to attain increased plasticity and flexibility, and thereby decreasing the possibility of cracking. A complete discussion of cracking and its behavior is given by Sherard (1963) [Ref. 7].

3.4.1 Core Thickness

There are no definite rules for evaluating core thickness. For practical purposes, one can consider the following factors [Ref. 7]:

1. The tolerable seepage loss.
2. The type of material available for the core and shells.
3. The design of the proposed filter layers.
4. The minimum width which permits proper construction.

All the above factors must be taken into consideration and finally engineering judgment is applied to select the minimum core thickness. The following criteria were presented by Sherard et al. (1963) which represent the rough cross section of the core [Ref. 7].

1. Cores with a width 30% to 50% of the water head have proved satisfactory for many dams, and they are adequate for any soil type and dam height.
2. Cores with a width 15% to 20% of the water head are considered thin but if designed and constructed properly filter layers are used.
3. Cores with widths much less than 10% of the water head have not been used widely, and require careful design and careful control.

Fig. 3.10 shows a cross section of the Tortoles Dam on the Nazas River in Mexico [Ref. 8], a slurry trench about 23 m deep was used to prevent leakage through an extremely pervious layer of sand and gravels with boulders. The bottom of the trench connected to a thick stratum of very dense and impervious silty sands and gravels. A core was constructed to provide more protection against possibility of leakage along the contact.

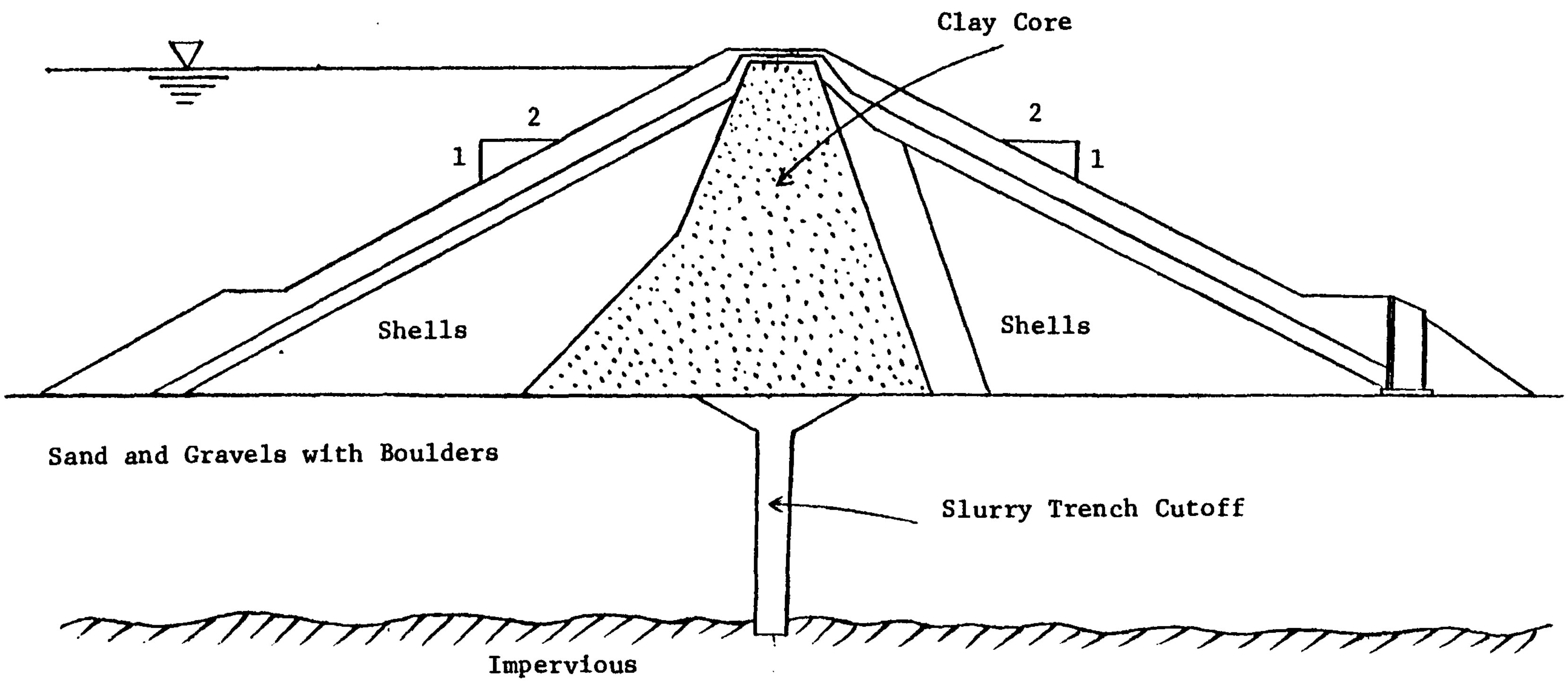


Fig. 3.10 Cross section of Tortoles Dam, Mexico [Ref. 8].

CHAPTER 4

STABILITY ANALYSIS OF EARTH SLOPES

4.1 General

Stability analyses provide a means of determining the factor of safety against slip for different earth slopes under various seepage and gravity loading conditions. The general objective is to search for a lowest possible factor of safety and perform further analysis and design based on this factor. The analyses are performed in order to evaluate the general condition of shear failure in which a portion of earth mass moves along a defined surface.

4.2 Slope Failure Characteristics

The movement of earth masses occurs in response to gravitational forces sometimes agumented by seismic activity. This results in the following main types of slope failures:

1. Rotational slides occur when soil masses change their position along failure surfaces approximating circular cylinders.
2. Translatory slides occur when earth masses move along natural planes of weakness, which are often horizontal.
3. Flow slides constitute the sudden movement of soil material, which results from weakened soil strength which in turn derives from slow or fast intrustion of water.

These movements occur primarily as a result of shear failure [the shear strength of the soil is exceeded by induced shear stresses

(Eq. 4.3)] at the boundaries of the moving mass. Typical modes of failures in dams are shown in Fig. 4.1.

Fig. 4.1.a shows a situation where foundation failure triggers a general dam failure, whereas Fig. 4.1.b shows failure in the upstream face due to a sudden drawdown situation. This type of failure is usually a result of sudden loss of effective strength brought about by high pore water pressures.

Fig. 4.1.c shows a general downstream slope failure. This type of failure happens usually because of excess seepage forces and loss of strength, due to excess pore water pressure and softened material structure.

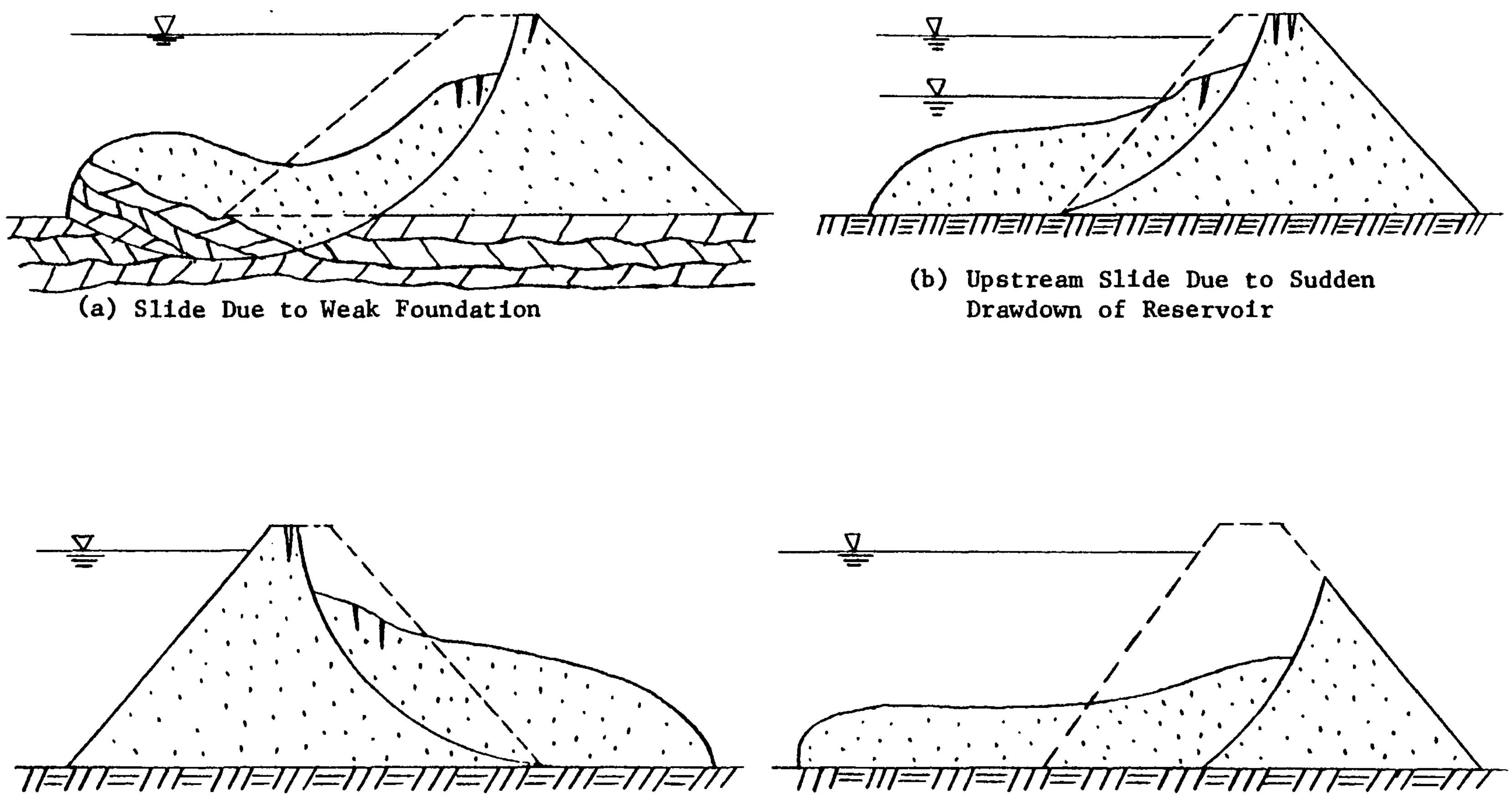
The general flow slide situation shown in Fig. 4.1.d is often brought about by violent actions such as earthquakes. Failures in such cases are not necessarily localized to only one arc surface.

The result of a earth mass slide can often be catastrophic, involving the loss of much property and many lives. To prevent such an occurrence, stability analyses, which follow a specific procedural pattern are used to evaluate the safety of the structure.

4.3 Factors Controlling Stability of Earth Dams

Usually failure occurs when the soil strength is decreased or an increase in soil stress is induced.

From an engineering point of view, the following factors, which have been described briefly in Chapter 1, control the stability of earth dams:



(a) Slide Due to Weak Foundation

(b) Upstream Slide Due to Sudden Drawdown of Reservoir

(c) Downstream Slide Due to Weak Embankment

(d) Flow Slide in Loose Fine Sand or Silt

Fig. 4.1 Modes of earth dam failure [Ref. 15].

1. Overtopping, results from inadequate spillway capacity. High speed surface flow creates large seepage forces which supersede the cohesion and friction and strengths of wash soils. To prevent this, the spillway should be designed for maximum flood.
2. Instability of slopes due to wave action, comes about due to high hydrostatic and hydrodynamic fluid pressure, which in turn results in loss of effective strength.
3. Instability of slopes due to internal seepage erosion or piping is a direct result of the pore water pressures influence on the effective soil strength.
4. Instability of the foundation may be brought about due to high subsurface pore pressure resulting in shear failure.

Failure of earth dams can result from any one of these factors, and in most cases involving slip, these features exist simultaneously. It is frequently difficult to determine which of the factors was the primary source for starting the failure mechanism and which factor(s) eventually caused total failure.

Another important factor contributing to stability is the slopes of the dam. The slopes of the dam must be determined by using the properties of the soil and the seepage pressures generated in the structure. The designer must insure the safety margin by selecting appropriate soil conditions and seepage criteria.

Adequate stability can be developed by decreasing the slope both upstream and downstream and by using erosion protection [Chapters 1, 2]. The slopes of an earth dam are rarely greater than 2 horizontal to 1 vertical [Ref. 7], and never larger than the earth mass' angle of repose.

4.4 Effects of Seepage on Slope Stability

Slope instability resulting from seepage is exhibited in two ways:

1. Removal of surface soil mass by seepage forces resulting in loss of overburden load which contribute to the total shear.
2. Development of seepage forces within the slope. Depending on the direction of the seepage vector with respect to gravity, the seepage forces may result in an increase or decrease in the effective normal stresses between particles. A reduction in effective normal stress will reduce the strength.

As discussed in Chapter 2, the flow of water through an earth mass is resisted by drag forces on the particles in the mass. The drag forces resulting from laminar flow are usually distributed uniformly or evenly throughout a unit volume of soil. The force per unit volume is designated as the seepage force and acts in the direction of flow with a magnitude of $i\gamma_w$. This is discussed in Chapter 2.

4.5 Effect of Rapid Drawdown on Stability

A rapid drawdown situation occurs when the upstream reservoir is lowered suddenly, or over short time span relative to the rate at which the excess or transient pore water pressure distribution in the dam can adjust to the changed conditions. This sudden change in the water level causes the upstream face to become a surface of seepage. The water flows out of the upstream face under a high gradient, leading to a dangerous stability problem in form of a possible slide.

To prevent such slides, the slope of the embankment must be designed to withstand the most unfavorable combination of high internal pore water pressure and drawdown in the reservoir.

4.6 Determination of Pore Water Pressure

The pore water pressure on the slip surface can be evaluated from the equipotential fields in a flow net, by subtracting the elevation head from the piezometric heads.

A typical flow net for the upstream part of an earth dam resting on an impervious foundation after a rapid drawdown is shown in Fig. 4.2.

The pore water pressure [see Fig. 4.3] can be evaluated as:

$$u = \gamma_w[h_c + h_r\,(1 - n) - h'] \tag{4.1}$$

where

γ_w = the unit weight of water

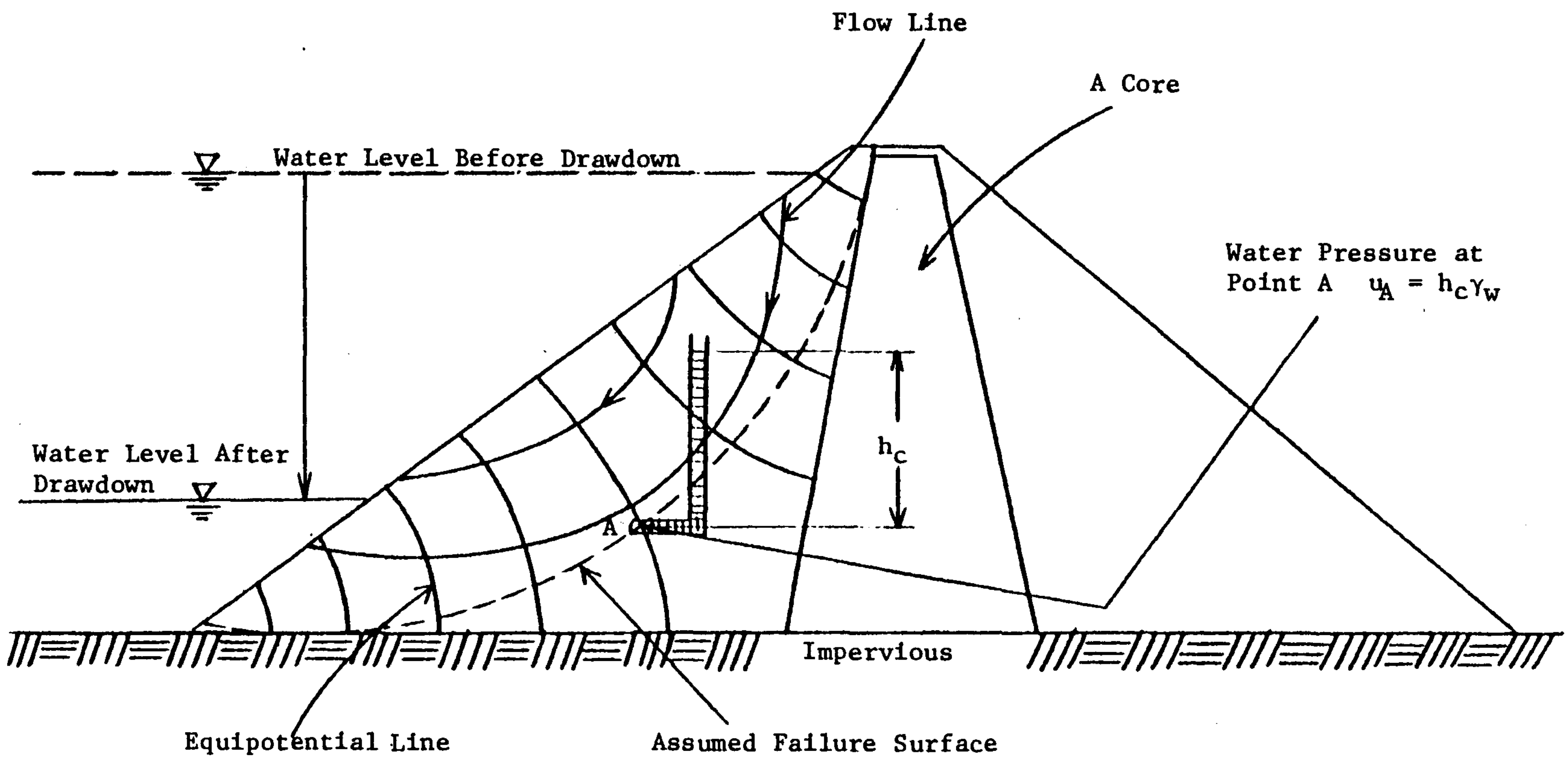


Fig. 4.2 Flow net in the upstream part of an earth dam after drawdown condition [Ref. 18].

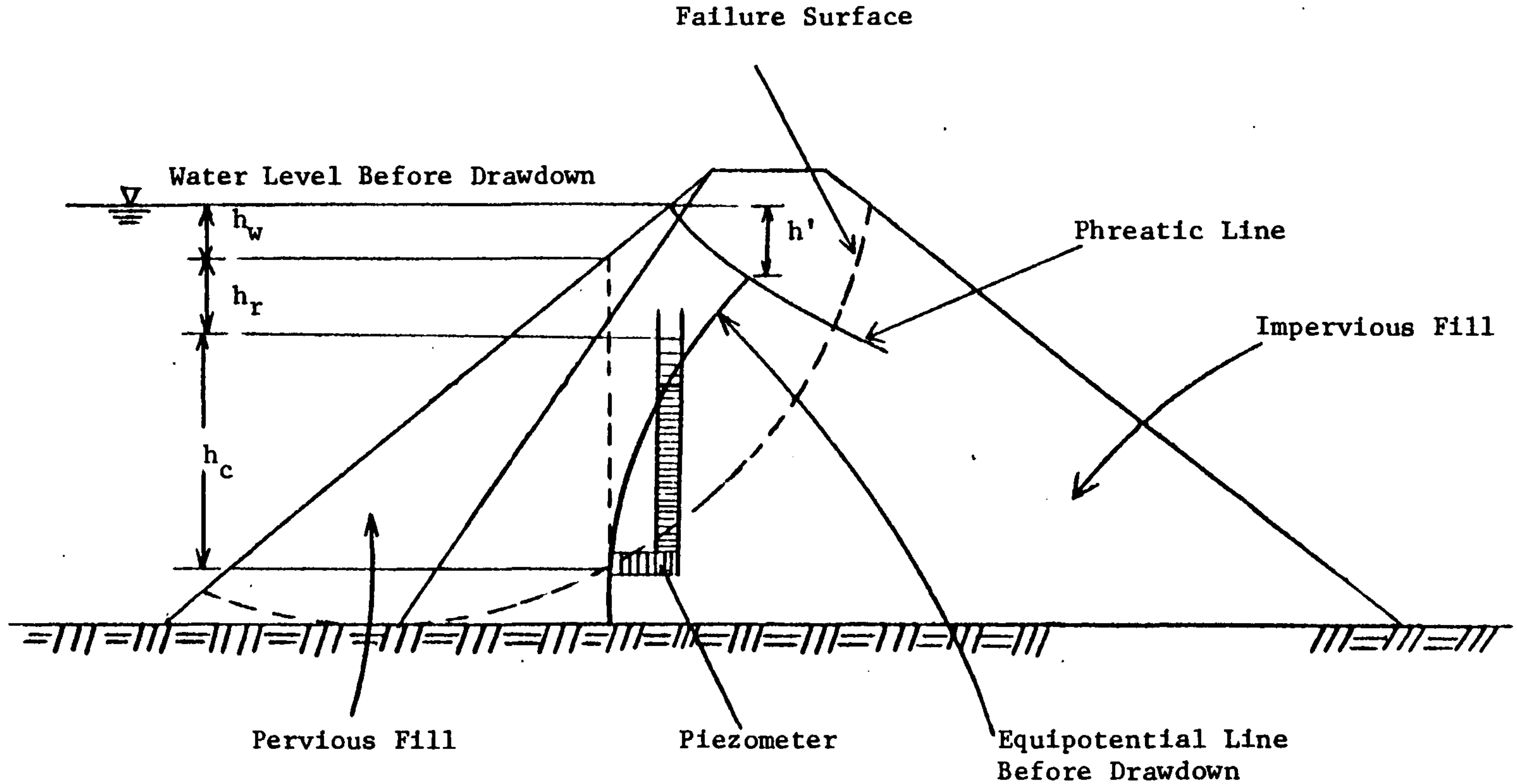


Fig. 4.3 Determination of pore pressure.

h_c = the vertical distance from the point at which the pore pressure is required to the upstream face of the impervious zone

h_r = the depth of the pervious material

n = the porosity of material

h' = the total head loss

An expression for evaluating pore water pressure due to an applied total stress increment which for example could occur during construction was suggested by Skempton [Ref. 19].

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \tag{4.2}$$

where

Δu = pore water pressure increment

A, B = pore water pressure coefficients [Ref. 19].

$\Delta\sigma_1$, $\Delta\sigma_3$ = increase in total principal stress.

The region near the core is usually saturated during construction, and the application of more overburden load will therefore result in an increase in the pore water pressure. This pore water pressure may in some instances overcome the total stresses, and the result will be a local failure during construction.

4.7 Stability Analysis

This section discusses the factor of safety and characteristics and concerns regarding analysis.

4.7.1 Factor of Safety

Generally any dam constructed must be designed and built in such a way that it is a safe structure, and the earth structure with its related facilities should be moreover protected from consequences of failure. The safety of a structural design is determined by evaluation of its "Factor of Safety." This factor is indeed the ultimate outcome of most stability analysis techniques. The factor of safety will cover some of the uncertainties associated with the measurement of soil properties, variation in geometry and material and also it is a measurement of protection against ultimate failure brought about by unanticipated environmental effects.

According to Casagrande, the factor of safety is usually expressed as a ratio between shear strength and induced shear stress [Ref. 4].

$$F = \frac{s}{\tau} \qquad (4.3)$$

where

F = factor of safety

s = available shear strength along shear surface

τ = shear stress.

This definition holds for one point in the soil mass. A broader definition follows later.

4.7.2 Methods of Analysis

Current methods of analysis for slope stability of earth embankments are based upon the assumptions and observations made earlier [Ref. 7]. Even though our increased understanding of certain aspects of stability problems, especially in the area of shear strength evaluation of soils and modern analysis techniques or criteria, have improved over the past decades, the basic assumptions remain the same.

The following assumptions are used in the stability analyses of earth embankments utilizing techniques such as limiting equilibrium [Ref. 7, 18].

1. Failure of an earth slope occurs along a particular, distinct and continuous slip surface.
2. The problem is considered to be two dimensional, although the real problem is three dimensional with a complex geometry.
3. The mass above the failure surface moves essentially as a rigid body when slip is incipient.
4. The shearing resistance of the earth mass at various points along the failure surface is independent of the orientation of the failure surface (the shear strength properties are isotropic).
5. The factor of safety is determined by the ratio of available shear strength to the induced shear stress [Section 4.7.1].

In performing stability analyses, there are various environmental loading conditions that may exist during the life of a dam. These are of a time dependent nature and they need to be evaluated with respect to the potential for a shear failure. These conditions can be classified as follows:

1. Short term condition.

   The short term analysis is employed to insure the stability during or immediately after construction. The soil parameters used need to reflect the fact that very little drainage or pore water pressure dissipation have occurred.

2. Long term condition.

   The long term analysis is employed to insure the stability of the structure after construction and during normal operation. The soil parameters used in this case should reflect that most excess pore water pressure has dissipated.

The design of earth embankments is usually governed by short term stability [Ref. 8]. However, analysis for long term stability should also consider steady state seepage and rapid drawdown situations. Softening and creep in the soil should also be considered.

4.7.3 Shear Strength

The shear strength parameters for the soil in the embankment and foundation are used to analyze the stability of the dam. These parameters are usually obtained from laboratory triaxial tests, conducted under various loading and drainage conditions.

In most methods of analysis, the computations may be made using either the principle of total stress or the principle of effective stress. These will be discussed in the following pages.

a) Total Stress Analysis

Total stress is the sum of the effective stress and the pore water pressure acting on any surface. Total stresses arise from loads and weights due to gravity. The pore water pressure is not considered in a total stress analysis. The total stress analysis is often used to determine the short term stability of embankments.

Undrained Shear Strength

When the earth mass is fully saturated, and the embankment is constructed quickly relative to the time of consolidation, the shear strength of fine grained soil does not change markedly with confinment. The angle of internal friction (ϕ) is often considered equal to zero. Since the shear strength of the soil is independent of the confining pressure, and constant, $s_u = c$ (undrained shear strength = cohesion) is applied. An unconfined compression test can often be used alone to determine the cohesion (c). Complete shear strength information, however, can be obtained from Unconsolidated Undrained (UU) triaxial tests. This will be discussed later. If the soil is partially saturated, a UU test will result in increased strength with increased magnitudes of confining pressure.

Unconsolidated Undrained Test (UU)

In this test, the confining stress (σ_3) and axial stress (σ_1) are applied so rapidly that there is no change in water content (pore water cannot escape from failure plane during the test) and there is no time for consolidation before the specimen has failed. The result of such a test is shown in Fig. 4.4.

b) Effective Stress Analysis

The effective stress analysis can be used to predict the long term stability of embankments. After some period of time the pore pressure becomes stable and easily measurable, and the effective stress can be evaluated and used for the analysis.

Consolidated Undrained and Drained Shear Strengths

In the effective stress analysis, the shear strength (s) of the materials are characterized by the Mohr-Coulomb failure criterion

$$s = c' + (\sigma_n - u) \tan\phi' \qquad (4.4)$$

where

s = shear strength on the surface at failure

c' = cohesion in terms of effective stress

ϕ' = angle of internal friction in terms of effective stress

σ_n = total normal stress acting on the failure surface

u = pore water pressure

σ_n' = effective normal stress acting on the failure surface ($\sigma_n' = \sigma_n - u$).

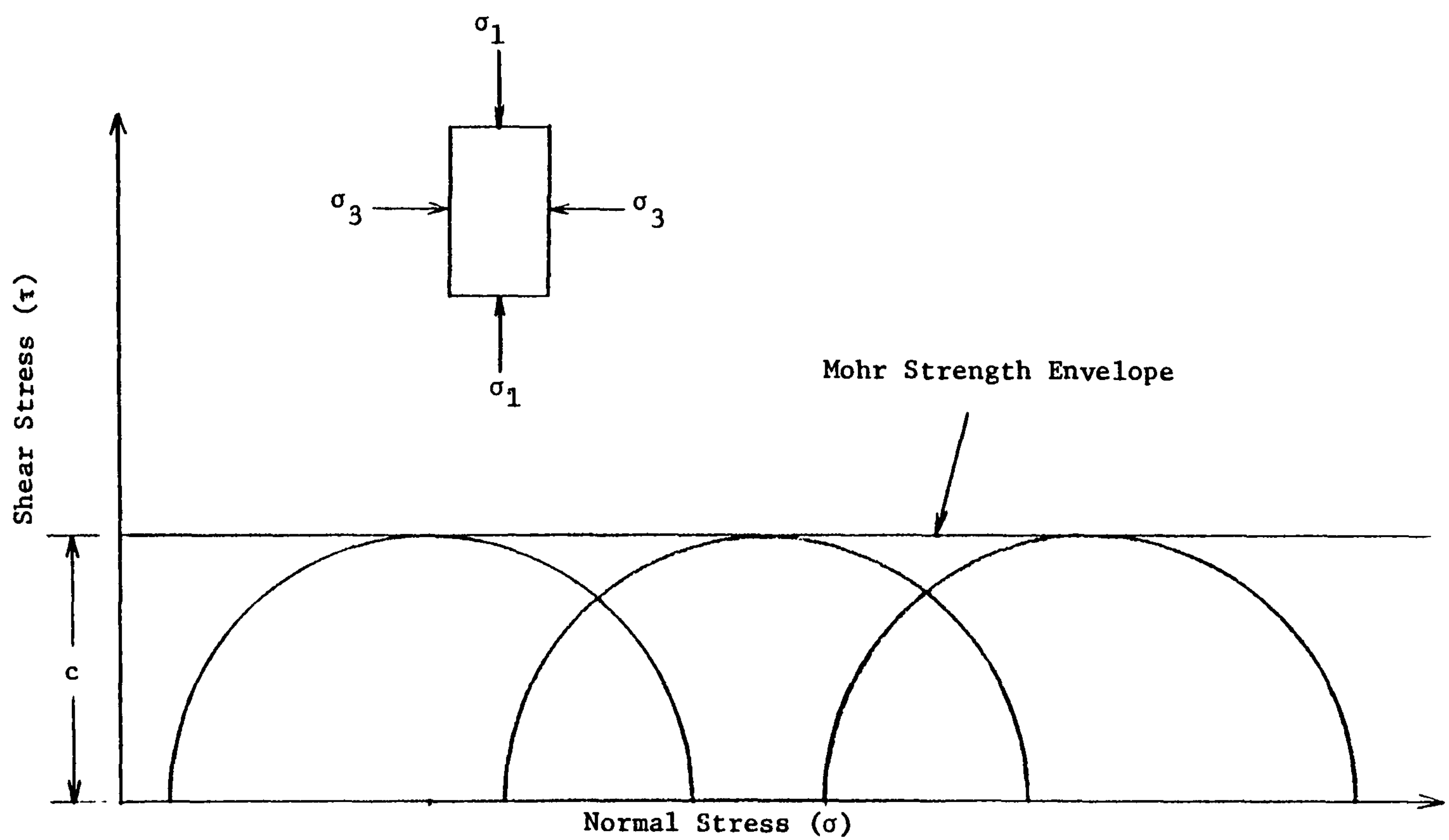


Fig. 4.4 Typical results of unconsolidated undrained triaxial test on cohesive soil.

Consolidated Undrained Test (CU)

In this test, the confining stress (σ_3) is applied slowly so that the pore water pressure does not change. The sample is consolidated fully under increase in effective stress (σ'). However, the axial stress (σ_1) is applied quickly and no drainage is allowed where shear occurs. This means that there is no further consolidation taking place [see Fig. 4.5].

If the soil is partially saturated, and pore water pressure is measured, the test will give the soil parameters in terms of effective stress (c' and ϕ').

Drained Test (D)

These tests are performed to evaluate the strength parameters in terms of effective stresses (c' and ϕ'). In this test, the confining stress (σ_3) and the axial stress (σ_1) are applied so slowly that there is no excess pore water pressure developed with the applied load. Complete consolidation is allowed during test [see Fig. 4.6].

The shear strength schemes are used by earth dam designers. Before the 1950's there were hardly any pore water pressure measuring techniques available [Ref. 7]. The main problem involved estimating the pore water pressure which would exist in the dam at various times, for use in the effective stress analysis. The total stress analysis was used more and considered to be the most realistic approach [Ref. 7].

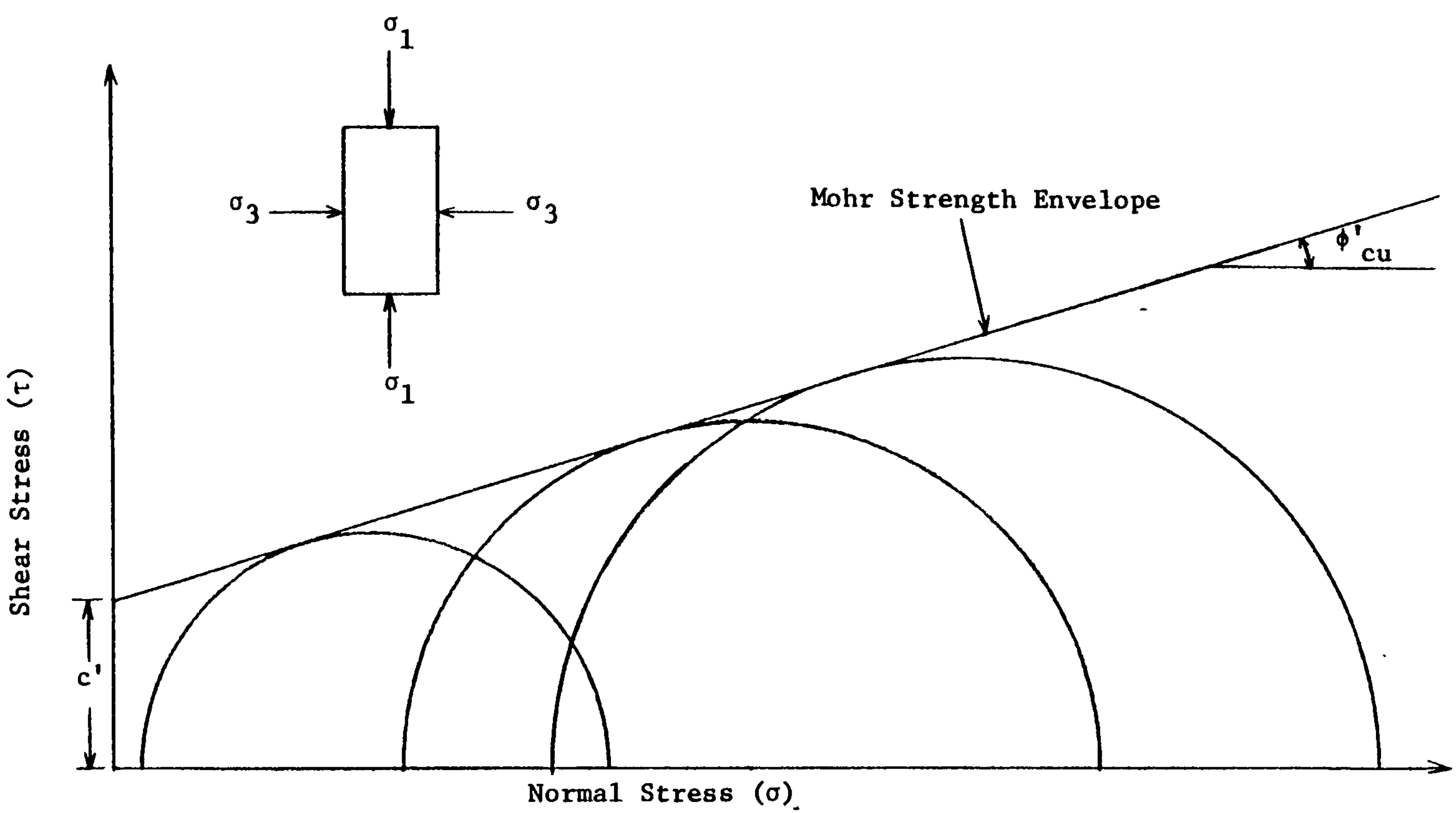


Fig. 4.5 Typical results of consolidated undrained test on saturated specimen.

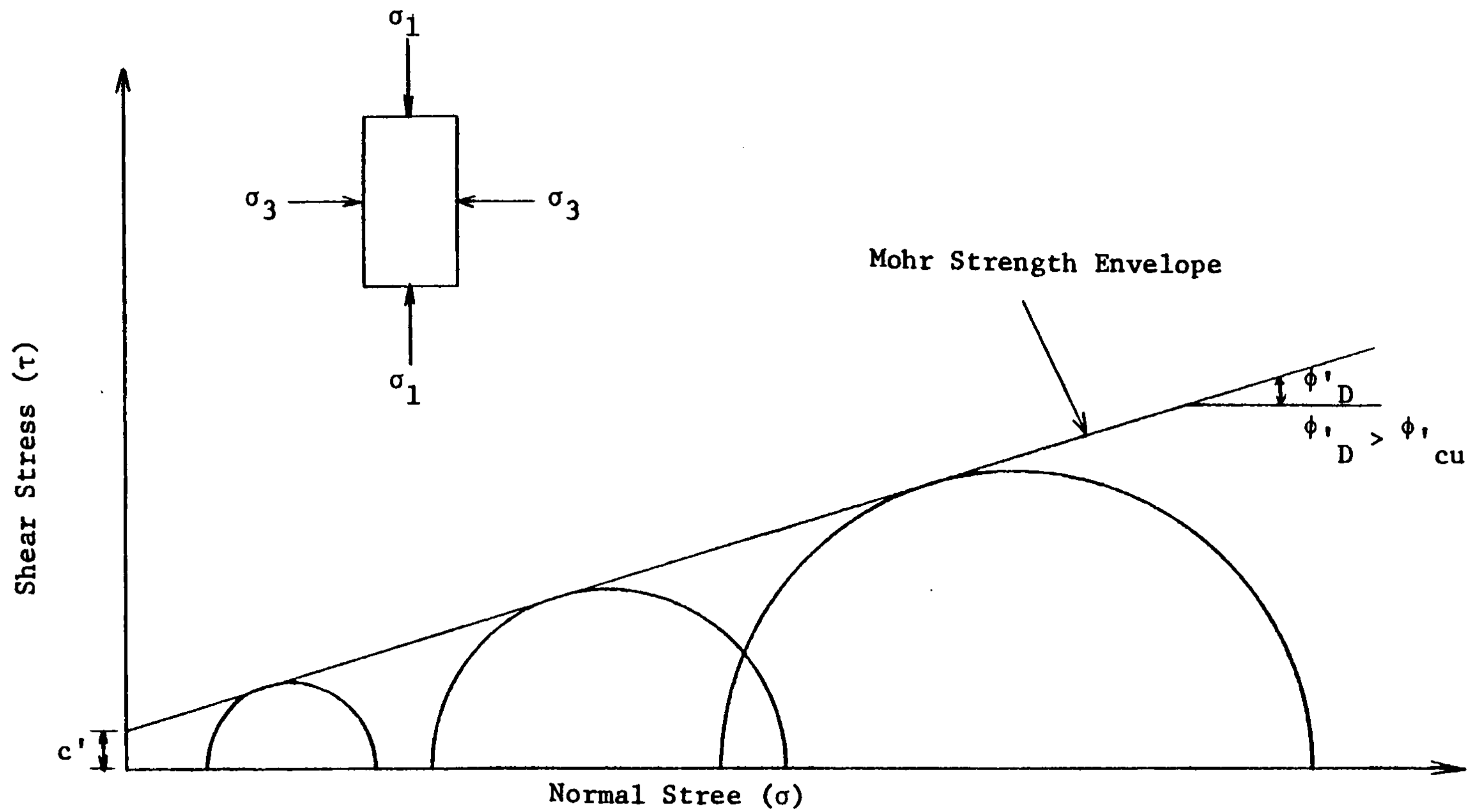


Fig. 4.6 Typical results of drained test on saturated specimen.

Today many engineers only use effective stress analysis. Others use the total stress method of analysis because they accept it as more reliable, conservative, and easier to use. According to Sherard (1963) [Ref. 7] both methods are reliable when applied with the same degree of understanding and judgment.

4.8 Methods Used for Stability Analysis of Embankments

Any embankment is subjected to stresses induced by its own weight. If the materials comprising the slope can develop enough shearing resistance with respect to induced shear stresses, the slope will remain stable. There are many different methods proposed for evaluating the stability of an embankment. Most of these methods are based on the shear strength of the soil and some specific assumptions with respect to the behavior of the embankment at failure. These methods can be used with effective stress (necessary to estimate the distribution of pore water pressure), or with total stress parameters (the shear strength is assumed to include the influence of pore pressure).

Many procedures have been used for analyzing stability problems. A few of these will be discussed here, and they are all limiting equilibrium schemes:

1. Standard method of slices.
2. Simplified Bishop method of slices.
3. Morgenstern and Prince Method.
4. Spencers method.

The computations in each method are laborious and the results vary greatly. Engineering judgment must be used in choosing the factor of safety. As a result of these problems, the analysis of slopes are performed with the aid of computer programs employing different methods for computing the factor of safety. All the analysis schemes discussed in the following are in terms of effective stresses.

4.8.1 Standard Method of Slices

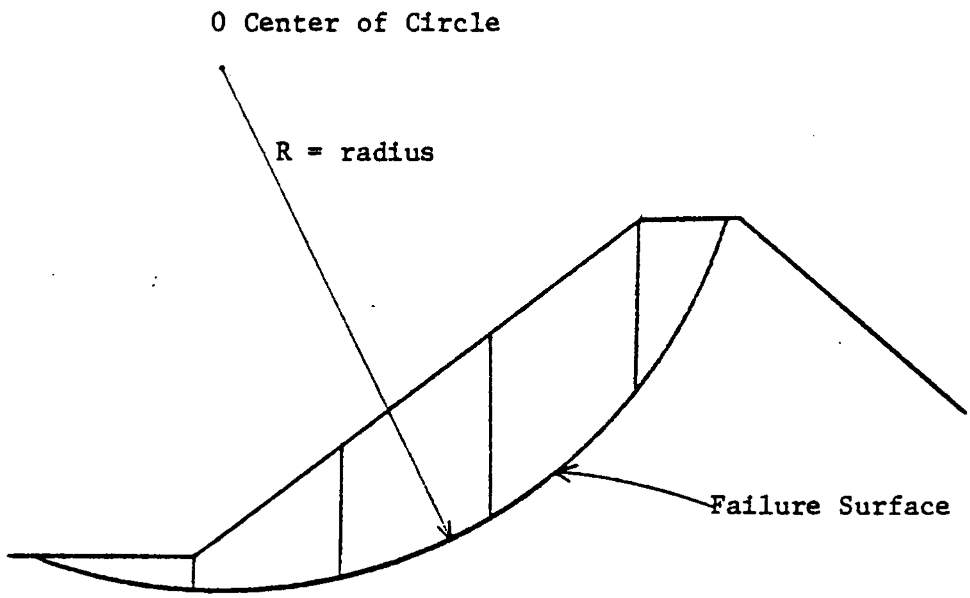
This method is one of the simpler schemes for analyzing embankment stability, and it has been adopted as one of the accepted methods of analysis by the U.S. Army Corps of Engineers [Ref. 8]. In this method it is assumed that the side forces acting on each slice have zero resultant in the direction normal to the failure surface. This situation is shown in Fig. 4.7.

The development of this procedure is referred to as Fellenius procedure [Ref. 17].

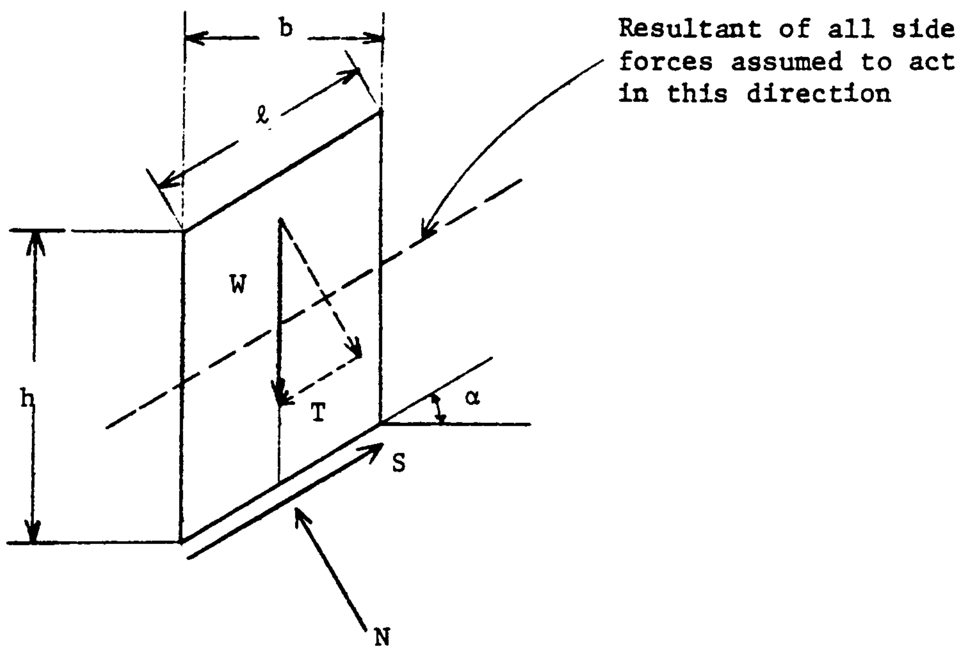
The procedure starts by assuming a circular arc surface of failure projected on a vertical section [see Fig. 4.7.a]. Then the sliding mass is divided into a number of slices. For each slice, the following forces are computed (Fig. 4.7.b):

1. The total weight (W); the area of the slice multiplied by its unit weight (γ_m)

$$W = b\ h\ \gamma_m$$



(a) Failure Surface is Approximated by Circular Arcs



(b) Forces Acting on Individual Slice

Fig. 4.7 Standard method of slices.

where

b = the width of the element

h = the height of the element.

2. The total normal force (N), acting on the bottom of slice

$$N = W \cos \alpha$$

where

α = the angle of inclination

3. The total shear resistance (S), acting on the bottom of the slice.

$$S = C + (N - U) \tan \phi'$$

where

C = total shearing resistance due to cohesion (c')

$$C = c'\ell; \ \ell = \frac{b}{\cos \alpha}$$

U = total hydrostatic forces acting on the bottom of the slice

$$U = u\ell$$

4. The total driving force exerted by each slice

$$T = W \sin \alpha$$

The procedure is derived by satisfying only one equilibrium condition, that is the equilibrium of moments about the center of the circular failure surface.

The factor of safety (F) for an assumed failure surface (circle) can be computed by:

$$F = \frac{\Sigma S}{\Sigma T} \tag{4.5}$$

or

$$F = \frac{\Sigma[C + (N - U)\ \tan\phi']}{\Sigma W \sin\ \alpha} \tag{4.5a}$$

After substitution for individual term in equation (4.5a) the factor of safety can be written in a different form as:

$$F = \frac{\Sigma[c'\ell + (W\ \cos\ \alpha - u\ell)\ \tan\phi']}{\Sigma W \sin\ \alpha} \tag{4.5c}$$

In order to satisfy the stability requirements, various centers and radii must be selected [see Fig. 4.8] and computations must be repeated until the arc which gives the minimum factor of safety is established.

4.8.2 Simplified Bishop Method of Slices

Bishop (1955) assumed that failure surface can be represented by a circular arc and it was also assumed that the side forces acting on any slice have zero resultant in the vertical direction [Ref. 17].

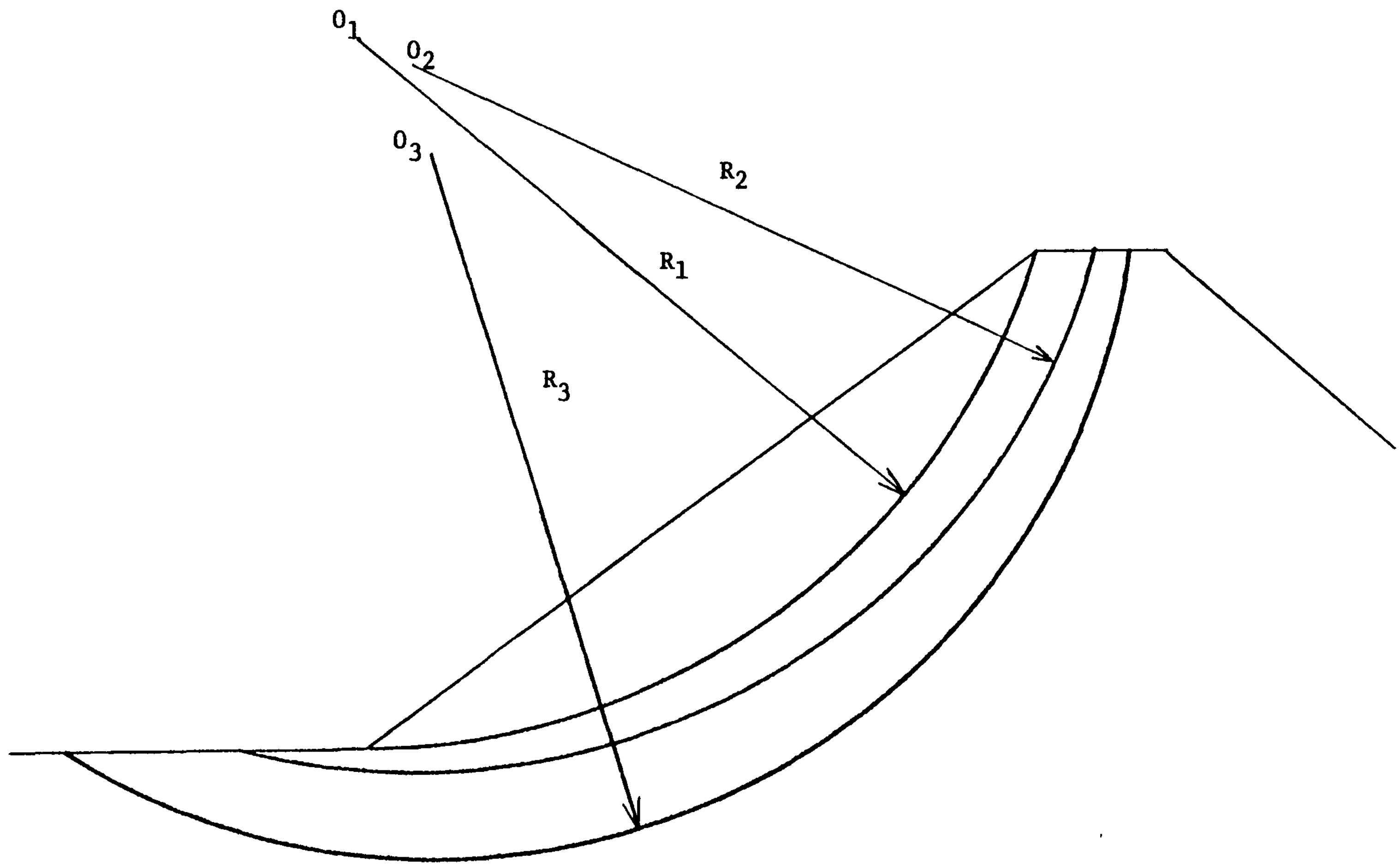


Fig. 4.8 Assumed failure surfaces used in analyzing the stability of embankments.

The equation for factor of safety is derived by satisfying two equilibrium conditions:

1. An overall balance of moments about the center of the circular slip surface.

2. A balance of forces in the vertical direction for each slice.

In order to obtain a statistically determinate solution, it is assumed that the forces act horizontally on the sides of each slice [see Fig. 4.9]. The forces acting on an element 3 are shown in Fig. 4.9 [Ref. 17].

Where

E_3 and E_4; the resultants of the total horiztonal forces on the sections 3 and 4

y_3 and y_4; the vertical shear forces

W; the total weight of slice of soil

P; the total normal force acting on the bottom of slice

P'; the normal effective force acting on the base of the slice

S; the shear force acting on its base

U; the total hydrostatic force on the bottom of the slice ($U = u.\ell$)

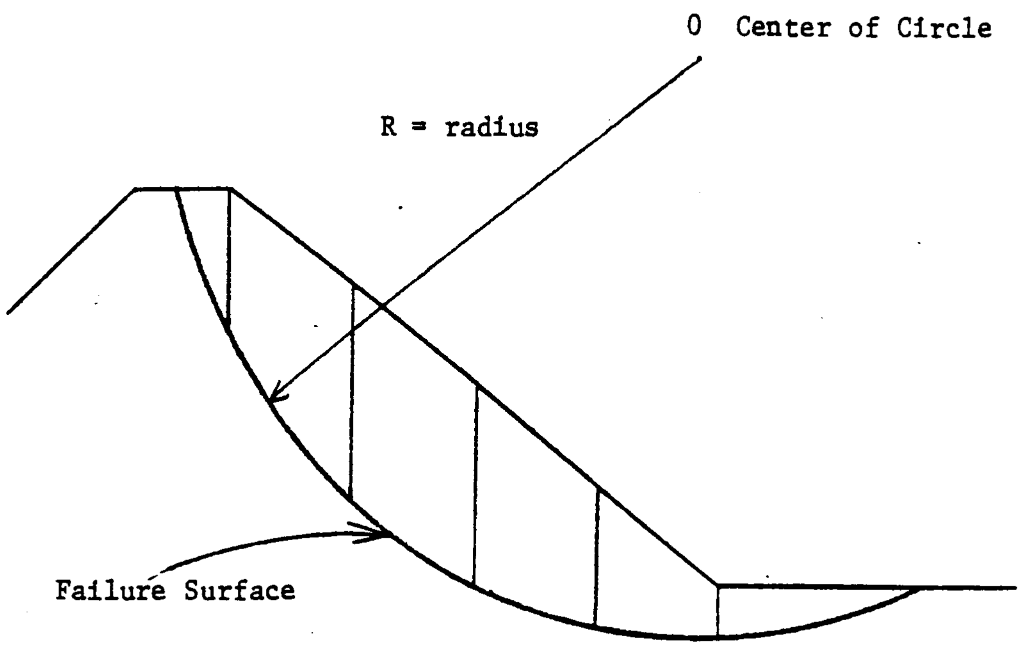
ℓ; the length of inclination

α; the angle of inclination of slice

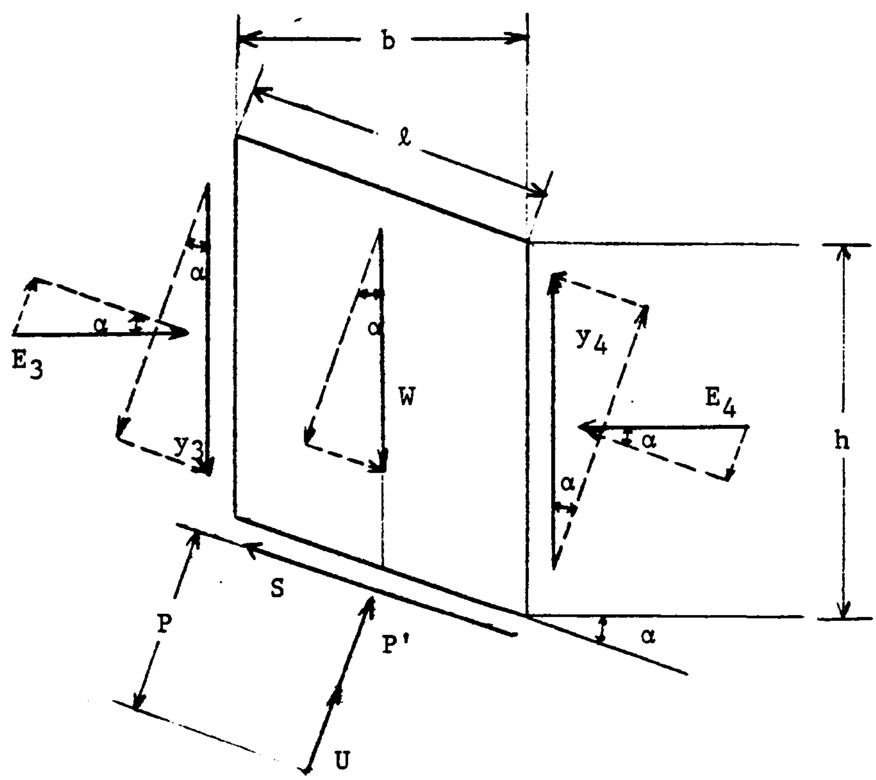
b; the width of the element

x; the horizontal distance from center of retation

h; the height of the element.



(a) Assumed Failure Surface



(b) Forces Acting on an Element (3)

Fig. 4.9 Bishop method of slices.

Bishop (1955) defined the factor of safety as the ratio of the available shear strength of the soil to that required to maintain equilibrium. The shear strength mobilized (S_m) is:

$$F = \frac{s}{S_m} \tag{4.6}$$

$$S_m = \frac{s}{F} \tag{4.4}$$

where

$$s = c' + (\sigma_n - u) \tan \phi'$$

$$\sigma_n = \frac{P}{\ell}$$

The magnitude of the shear strength mobilized to satisfy the conditions of equilibrium can be determined by rearranging the above equations, we get:

$$S_m = \frac{1}{F} [c' + (\frac{P}{\ell} - u) \tan\phi'] \tag{4.7}$$

The shear force (S) acting on the base of the slice can be expressed by:

$$S = S_m \ell \tag{4.8}$$

Substituting equation (4.7) into equation (4.8) and simplifying, we can obtain an expression for shear force (S) in terms of the Mohr Coulomb relationship for limiting equilibrium:

$$S = \frac{1}{F} [c'\ell + (P - u) \tan \phi'] \tag{4.9}$$

where

$$P = P' + u\ell$$

The normal effective force (P) as shown in Fig. 4.9.b is found by satisfying the equilibrium equation in the vertical direction ($\Sigma F_y = 0$) and assuming that the side forces (y_3, y_4) have zero resultant ($y_3 - y_4 = 0$). Thus,

$$P' = \frac{W - \ell\,(u \cos\alpha + \frac{c'}{F} \sin\alpha)}{\cos\alpha + \frac{\tan\phi' \cdot \sin\alpha}{F}} \qquad (4.10)$$

The expression for factor of safety (F) is obtained as:

$$F = \frac{\Sigma[c'b + (W - ub)\tan\phi'] \dfrac{\sec\alpha}{1 + \dfrac{\tan\alpha \cdot \tan\phi'}{F}}}{\Sigma W \sin\alpha} \qquad (4.11)$$

Equation (4.11) requires a trial and error solution procedure since F appears on both sides of the equation.

Once the factor of safety has been evaluated, the normal force ($P = P' + u\ell$) acting on the bottom of each slice can be determined. The magnitude of the normal force is very useful for evaluating how reasonable a solution for factor of safety can be since no unique solution exists.

4.8.3 Morgenstern and Price Procedure

The Morgenstern and Price (1965) procedure satisfies all requirements of static equilibrium. This procedure assumes that the vertical and horizontal side forces on individual slices are not zero. The solution procedure in this method involves calculating a factor of safety (F), normal forces (N), the components of side forces (E and x), and the coordinate of side forces (y_t) [see Fig. 4.10] [Ref. 14]. The solution for factor of safety (F) and unknown constant (λ) requires a trial and error procedure. A complete discussion of this procedure which is very involved is given in various textbooks [see Ref. 14, pp. 13-18 and Ref. 20, pp. 79-93].

4.8.4 Spencer's Method

Another method of analysis for determining the stability of an embankment has been described by Spencer (1967) [Ref. 21]. This method satisfies the conditions of equilibrium for both forces and moments by assuming that the resultant of the side forces are parallel. The forces acting on a typical slice is shown in Fig. 4.11 [Ref. 21]. The symbols used in Fig. 4.11.b can be described as follows:

W = the weight

P = total normal force acting on the base of the slice and it has two components:
1) the force P' due to effective stress
2) the force U due to pore water pressure

S = shear force

Z = the inter-slice forces

Q = resultant of pair of inter-slice forces

α = inclination angle

θ = slope of resultant (Q) of pair of inter-slice forces.

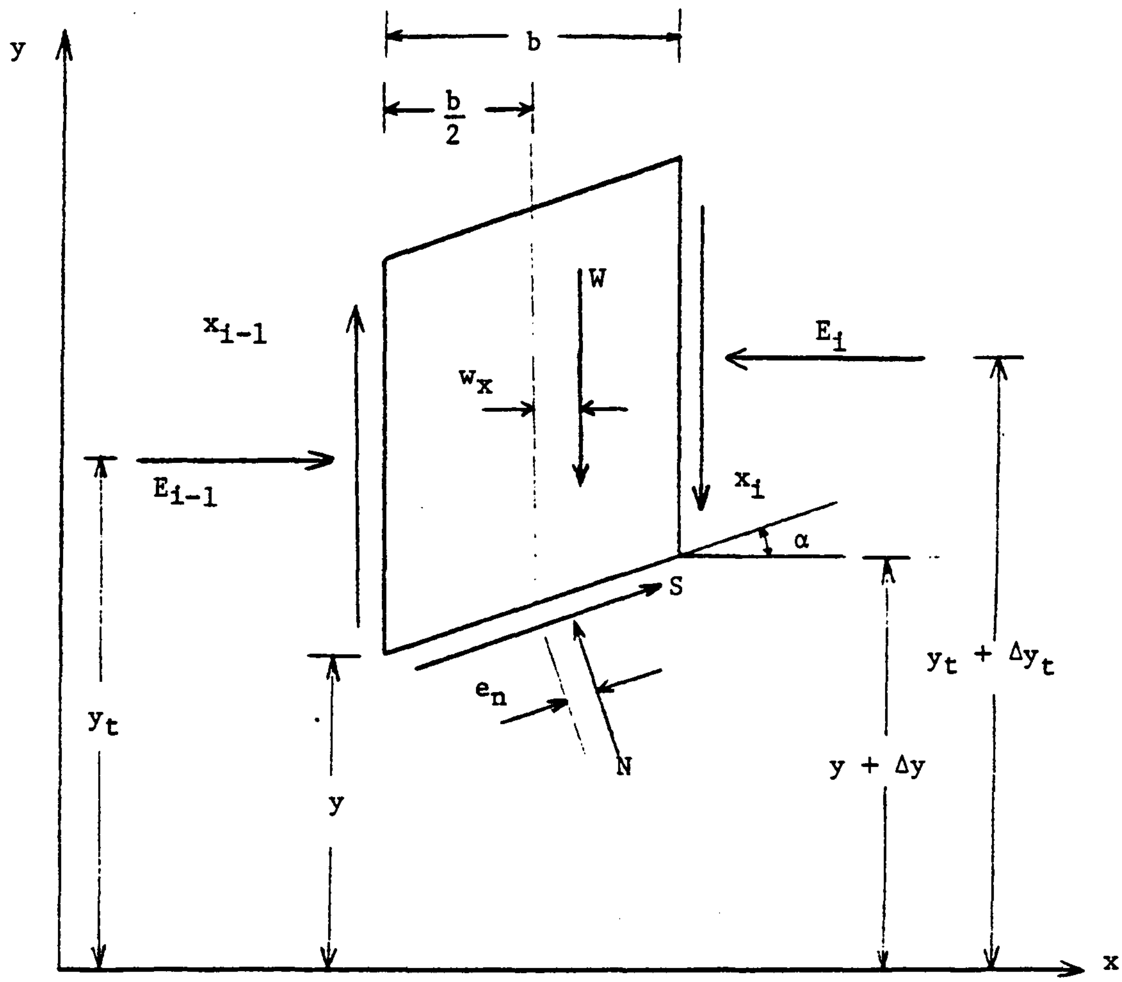
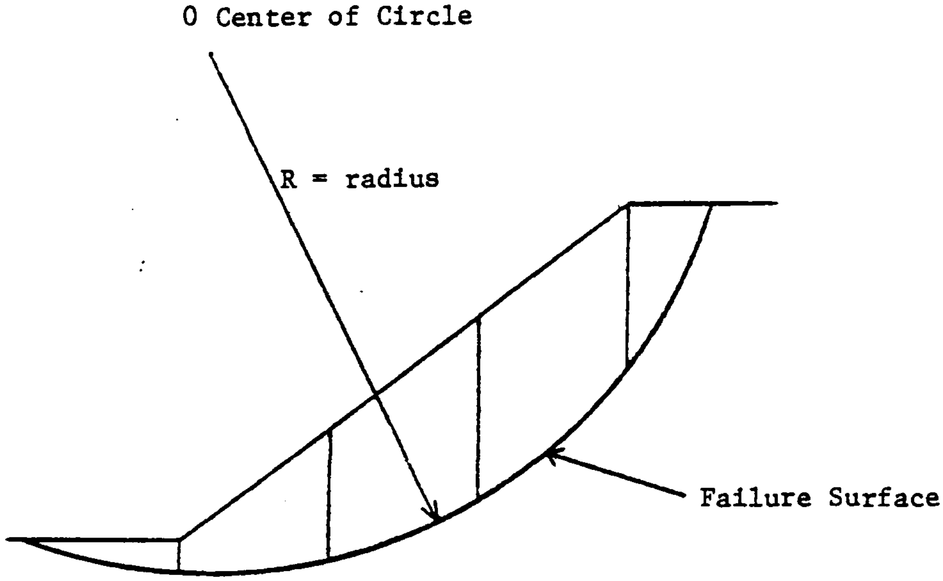
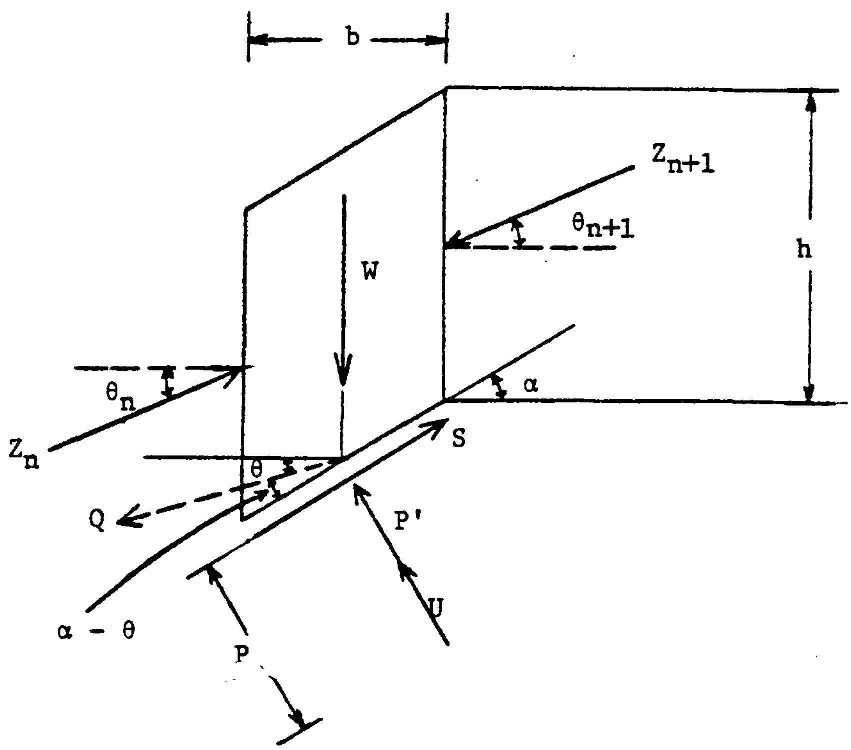


Fig. 4.10 Morgenstern and Price procedure (forces and coordinates for a typical slice) [Ref. 14].



(a) Assumed Failure Surface



(b) Forces Acting on Individual Slice

Fig. 4.11 Spencer's method [Ref. 21].

Spencer (1967) did indeed prove that the inter-slice forces are parallel (θ = constant), and as a result, the requirement for equilibrium of forces in the horizontal and vertical directions can be obtained as:

$$\Sigma Q = 0 \tag{4.12}$$

In other words, the sum of the horizontal and vertical components of the inter-slice forces need to be zero in order to satisfy equilibrium.

To determine an equilibrium equation for moments Spencer assumed that the lines of action of the weight (W) and normal force (N) passed through the center of the base of the slice [Ref. 14]. Therefore the forces (W, N, S) would produce no moment about the center of the base and the only remaining force is Q [see Fig. 4.11], which act through center base of the slice to satisfy moment equilibrium and as a result:

$$\Sigma Q \cos(\alpha - \theta) = 0 \tag{4.13}$$

The resultant for the side force (Q) can be derived by consideration of the requirement for equilibrium of forces and is given by:

$$Q = \frac{\{[c'\ell + (W \cos\alpha - u\ell)\tan\phi']/F\} - W \sin\alpha}{\cos(\alpha - \theta)[1 + \frac{\tan\phi'}{F} \cdot \tan(\alpha - \theta)]} \tag{4.14}$$

Values of factor of safety (F) and inclination angle (θ) can be determined by trial and error procedure and must satisfy all three equations.

All of the methods of analysis discussed involve evaluating a number of unknowns from equilibrium point of views. A summary of the assumptions used and the unknowns, which are calculated in different schemes, is given in Tables 4.1 and 4.2.

TABLE 4.1

Assumptions in equilibrium slope analysis methods

Standard Method of Slices

a) Weight (W) and normal (N) forces coincide through the base of the slice.

b) No side forces.

Simplified Bishop Method

a) Weight (W) and normal (N) forces coincide through the base of the slice.

b) Side forces are horizontal.

Morgenstern and Price Method

a) Side forces follow a pattern.

b) The side force inclinations are varying linearly across each slice.

Spencer Method

a) Weight (W) and normal (N) forces coincide through the base of the slice.

b) Side forces are parallel.

TABLE 4.2

Unknowns calculated in equilibrium analysis methods

Standard Method of Slices

a) Factor of safety (F).

Simplified Bishop Method

a) Factor of safety (F).

b) Normal forces (N).

Morgenstern and Price Method

a) Factor of safety (F).

b) Normal forces (N).

c) Side forces inclination factor (λ).

d) Horizontal side forces components (E).

e) Side force positions (y_t).

Spencer Method

a) Factor of safety (F).

b) Normal forces (N).

c) Side force inclination (θ).

d) Side force resultants (Z).

e) Side force positions (y_t).

CHAPTER 5

CONCLUSIONS

The failures of a number of earth dams in the recent past have warranted engineers to analyze the seepage and the stability problems of dams more thoroughly and critically.

The various factors responsible for the failures have been dealt with in detail in Chapter 3. Many remedial measures have been suggested to control seepage. Some of the methods can be used in combinations with others, depending on the geometry of the dam, the properties of materials used and the hydrostatic data.

Before any of these schemes are selected, it is very necessary to carefully study the geological behavior of the foundation and the material used for construction of an embankment. This will assist the engineer to select proper design criteria for preventing the seepage problems occurring in the embankment and its foundation. In addition to these features, the designer must make sure that the dam is stable throughout its operational life. Long term and short term stability analysis should be considered. This can be achieved by using the methods discussed in Chapter 4. All of the schemes employed for analyzing the stability of the embankment yield reliable results. Since the standard methods of slices and others involve a lot of hand calculation and have certain limitations in regard to finding out the realistic failure surface, they may not give accurate results; therefore, computational techniques, such as, finite element and finite difference methods etc. may be implemented which require the use of computers. This will assist the designer to choose the realistic failure surface which gives the lowest factor of safety for the structure. It must be kept in mind that the greatest uncertainty in stability problems arise in the proper selection of shear strength. Careful judgment in the selection of strength parameters must be applied in the stability analysis of earth structures.

To determine the values of shear strength of various materials involved in the construction of dams, it is desirable to select a number of random samples from various sections of dams during construction. The test results should be critically analyzed and carefully interpreted to arrive at representative values of strength parameters.

The results of the seepage and stability analyses must be compared with actual values observed by hydrostatic pressure measurements during and after construction. This can be achieved by using different types of installations such as piezometers, etc. installed at various points in the foundation and embankment, depending upon the observed behavior of dams, it may sometimes be necessary to modify the design of dams during construction or apply some additional preventive measures during operation to ensure the safety and stability of the dam.

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SEEPAGE AND STABILITY OF EARTH DAMS

by

Khosrow Mahgerefteh

(ABSTRACT)

In general, all water retaining structures are subjected to seepage through or under their embankments. If the seepage is not controlled, structural failure is certain.

The object of this project is to present a review of the seepage problems and stability considerations involved in the analysis of earth dams. It contains a review of the seepage characteristics of soils and their effects on earth embankments. The problem resulting from seepage and different methods for controlling seepage are fully discussed.

Because many uncertainties remain, the stability of the embankment must be determined. Four different methods are presented for analysis of earth embankments to determine their factor of safety against stability failure.