STUDIES OF UPSTREAM BLANKETS FOR CONTROL OF UNDERSEEPAGE IN EARTH DAMS

M.Sc THESIS

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CERTIFICATION

This project thesis entitled "Studies of Upstream Blankets for Control of Underseepage in Earth Dams" by Simeon Ohiani Yeesuf, meets the regulations governing the award of the degree of M.Sc. (Water Resources and Environmental Engineering) of Ahmadu Bello University and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This thesis is dedicated to my wife

Mrs. Veronica Anno Yeesuf

for her forebearance, impressionable loyalty,
incredible capacity for hard work and a good heart.

DECLARATION

I. hereby declare that this thesis has been written by myself and that it is a record of my own research work at Ahmadu Bello University, Zaria. It has not been accepted in any previous application for a higher degree.

All sources of information have been specifically acknowledged in the reference section.

30/11/87

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ABSTRACT

Experimental investigations have been carried out into the efficiency of a blanket design in controlling under-seepage. Also, accurate numerical method for the mathematical analysis of the blanket designs for efficiency are proposed.

In the experimental investigations, use was made of a seepage tank model under steady flow regimes. The blanket design in each case Was constructed of clay. Five different design shapes are considered in the study. They are; rectangular, triangular and trapezoidal blanket shapes. Others are composite blanket shapes the first of which is the stepped-rectangular and lastly the rectangular-trapezoidal blanket. The proposed empirical equations are obtained using Bessel's functions of the first and second kinds respectively. The final equations are simple and can be readily used for design calculations.

The theoretical results are tested against the experimental data, with accurate and physically plausible results. Both results show that the triangular blanket is more effective than the rest .blanket design shapes.

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LIST OF SYMBOLS

A	:	Area (cm^2); constant of in tegration
a	:	Diameter (cm); area (cm^2)
B	:	Constant of integration
b	:	Abbreviated notation for $(k_b/k_f Tt)^{\frac{1}{2}}$, dimension (cm)
C	:	Constant in Bessel function of the first kind
D	:	Constant in Bessel function of the second kind; diameter (cm); distance (cm)
D_{15}	:	15 per cent finer by weight than the grain size
D_{85}	:	85 per cent finer by weight than the grain size
F	:	Uplift force per unit width of dam (N)
F_o	:	Uplift force per unit width of dam in absence of blanket (N)
F_R	:	Reduced uplift force per unit width of dam (N)
G_s	:	Factor of safety against piping
H	:	Reservoir head (cm); piezometric head (cm)
h	:	Head across the dam (cm)
h_x	:	Local uplift pressure head at a location X
i	:	Mean hydraulic gradient (=h/l)
$J_o(\ )$	:	Modified Bessel function of the first kind and of the zeroth order
$J_1(\ )$	:	Modified Bessel function of the first kind and of the first order
k	:	Coefficient of permeability (cm/s)
k_b	:	Coefficient of permeability of the blanket material (cm/s)
k_f	:	Coefficient of permeability of the foundation material (cm/s)
L_o	:	Projected length of the trapezoidal blanket (cm)
L_1	:	Length of the upstream blanket (cm)

$(L_1)_R$: Length of the rectangular blanket (cm)

$(L_1)_T$: Length of the triangular blanket (cm)

L_2 : Distance between upstream end of blanket and the toe of the dam (cm)

L_d : Length of the base of the dam ($=L_2-L_1$), (cm)

l : Length of the soil sample (cm)

Q_o : Seepage discharge per unit width under the dam without any blanket ($cm^3/s/cm$)

Q_m : Underseepage discharge per unit width of dam ($cm^3/s/cm$)

Q_x : Underseepage discharge per unit width at any location.

q_o : Discharge per unit time (cm^3/s)

S : Slope of the blanket top surface

T : Thickness of the pervious foundation (cm); coefficient of transmissibility (m^2/s)

t : Thickness of the blanket at the impervious section of the dam (cm); time (s); temperature (°C)

U : Ratio ($=X_n/b$)

W : Soil unit weight (KN/m^3); width of aquifer (m)

X : Distance from upstream end of blanket toward the dam (cm)

X_n : Distance (cm)

$Y_o(\ )$: Modified Bessel function of the second kind and of the zeroth order

$Y_1(\ )$: Modified Bessel function of the second kind and of the first order

y : $h-h_x$ (cm)

Z : Abbreviated notation for $(y/\sqrt{X})$

α : Abbreviated notation of $(k_b/k_fT, (m^{-1})$

γ : Specific weight of water (kN/m^3)

ϕ : Velocity potential

τ : Abbreviated notation for $2({}^{a}L_1/S)^{\frac{1}{2}}$

τ_o : Abbreviated notation for $2({}^{a}L_o/S)^{\frac{1}{2}}$

θ : Abbreviated notation for $\sqrt{X}$, $(cm^{\frac{1}{2}})$

ψ : Abbreviated notation for $(k_b/k_f T\Psi)^{\frac{1}{2}}$. (cm^3/cm)

Ψ : Volume of blanket material per unit width, (cm^3/cm)

CHAPTER 1

INTRODUCTION

1.1 The importance of a dam

One of the most important problems facing the world today especially in the developing countries is the provision of adequate supplies of fresh water for industry, for agriculture and indeed for the continuance of basic activities. Inherent in this problem however is the incidence of waste, one serious aspect of which is the loss of valuable water carried regularly to the sea in floods. In all water projects, the ultimate objective must surely be to conserve and control all fresh water in order that what is so freely given by nature should be used efficiently and effectively for the future wellbeing of mankind. It is against this background that scientists and engineers embarked on dam constructions since the earliest times. Since then there has been a marked increase in expenditures for projects involving dams. This increase has led to an intensification of experimental research and a reexamination not only of details and methods of construction but also of many of the theories of design. All of these have resulted in a substantial improvement in the art and science of dam building.

Dam building is however a challenge. Starting with the construction of "The Saad-el-Kafara" dam near Cairo 4,800 years ago, building of a dam is still a challenge today. It is fundamental for the engineer to realise that no matter how strong he may build a dam there will always remain some risk - however small - that unforeseen forces may cause it

to fail. He must recognize the various undesirable events that may occur, and assess the probability of their simultaneous occurrence. To accept the possibility of some danger under the worst combinations may be both acceptable and economical but the design engineer must understand the risk he takes. The failure of a dam is usually catastrophic, hence the responsibility of the design engineer is great. Historically, most dam failures are usually due to seepage piping and sliding. In the other cases, failures may result from overtopping of existing spillways or waterlevel control facilities.

1.2 Foundations and seepage

The term "foundation", as used herein, means all of that part of the area under and adjacent to the dam which in any way will affect or be affected by loading, scour, or leakage. A good foundation is of ample strength to withstand the weight of the structure and to prevent sliding. When made tight enough, it can prevent excessive leakage and reduce uplift as much as possible. Seepage, which occurs within the foundation and abutments, results in water loss by leakage, uplift forces at the bottom of the dam, and possible creation of instability in foundation (and abutment rocks or soils). In some instances, it can lead to the development of destructive high-velocity concentrated flows by piping and cavitation. A limited amount of slow seepage through and below a dam is permissible, but every effort must be made to control and prevent concentrated seepage flows that will threaten the life of the foundation or the dam above it.

Unlike investigations for long underground tunnels, investigations of proposed dam sites are concentrated within relatively small areas. Because of the threat to human life and property posed by failure of foundations and subsequently the dams, careful examinations of foundations conditions and the geology of the reservoir site prior to design and construction is necessary. In general, failures in flat, nearly horizontal foundations of earth and rock-fill dams are not the result of shearing dislocations owing to the load of the dam. Instead, foundations give way because of inadequate treatment of seepage, either within the dam (which is not touched upon in this thesis) or beneath the dam. This seepage which occurs beneath the dam or within the foundation is termed underseepage. This forms the subject matter of the thesis. Underseepage could also occur as a consequence of a construction of a dam on a foundation which slopes steeply upstream or downstream. Discussing the importance of foundation in seepage control, Terzaghi (1929) observed that

> To pass judgment on the quality of a dam foundation is one of the most difficult and responsible tasks. It requires both careful consideration of the geological conditions and the capacity for evaluating the hydraulic importance of the geological facts which can only be obtained by a thorough training in the hydraulics of seepage.

In the design of earth dam built on pervious foundation, there appears to be a trend toward the control of underseepage by construction of an upstream blanket connected to the impervious section of the dam. In dams where the control of underseepage is undertaken primarily to increase the stability of the structure, rather than to reduce seepage losses to a minimum,

this design offers several advantages over a central impervious section in the dam, and a cutoff through the foundation. The advantages include: (1) It helps to provide enough frictional resistance to prevent the dam from sliding downstream on its foundation under the reservoir pressure; (2) By connecting it to the upstream face of the dam possible seepage at the base of the dam is reduced which results in greatly reducing the uplift and (3) It permits cutoff to be placed at some distance from the dam as in the case of natural slope of many bedrocks, the combined result of which is the prevention of underseepage flow, uplift forces and the reduction of exit gradient. From the standpoint of structural stability and water storage conservation, control of underseepage is of critical importance in an earth dam. The construction of an upstream blanket connected to the impervious section of the dam and a downstream blanket with drainage facilities is a common and technical practice. It offers a better solution from a structural and economic viewpoint.

There have been many proposed solutions to the engineering problems presented in the use of blankets. In each case, blankets of uniform thickness were assumed in order to simplify the mathematical problems involved. Bennett (1946) for example, attempted the use of a triangular blanket on the upstream side in that such an arrangement would offer greater resistance and would be more efficient but could not support the claim with adequate mathematical proof. By means of model experiments and mathematical analysis, this thesis seeks to inquire into the most efficient shape of the blanket that will solve the problem of underseepage.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The orign of study of seepage phenomena in porous media goes back to Darcy (1856), who carred out experiments about the flow of water in pipes filled with sand and established the steady law of this movement. The professional literatur on seepage embraces many fields of interest. Since 1920, many comprehensive bibliographies of seepage have appreared which provide brief quatations of literature world wide. The rigorous mathematical theory of the theory of the flow of ground waters under hydraulic structures was founded by Pavlosky (1922). It is also credited to him to have conducted the first investigations about the application of the method of inverses to the theory of unconfined flows. Of great merit is Pavlosky's application of the experimental method of the electro - hydromechanical analogs (Egda), which at present are widely used in scientific research.

In 1949, Zjoukovsky applied a special mapping technique to solve the problems of unconfined flow situated on deep foundation. Kochina - Polubarinova (1952) correlated the exit point of the free surface to the characteristic dimensions of the section in unconfined shallow - foundation flows. Dupuit and Boussinesq (1962) gave the hydraulic theory of the flow of ground waters. Also, 1962, Forchheiner developed the hydraulic theory of wells. In 1980, Gill gave the theory of blanket design for dams on pervious foundations.

Seepage, actually, is a general term describing the movement of water through the ground or other porous media to the ground surface or surface water bodies. The term is well established in the engineering literature in connection with groundwater movement, from and to surface water bodies, particularly where associated with structures such as dams, canals and levees. It can occur to lower lying lands from normal water courses, and/or the sea; from artificial water reservoirs, underneath various hydraulic structures and from leakages of artesian wells. This however should not be confused with a spring, which is a concentrated discharge of groundwater appearing at the ground surface as a current of flowing water. In the limiting case of dam engineering, one can consider seepage through the dam (which we do not touch upon) and the seepage through the foundation or beneath the dam, called underseepage.

2.2 Mechanism of Underseepage

In the total history of dam construction certainly many hundreds have failed. There are no accurate records of most of the failures and the extent of the damage that resulted from torrential inundation of lower areas as a consequence of their failures. Disasters owing to dam failure for which careful documentation is available indicate a variety of causes within the foundation. The Frazier Dam, Idahosa, failed in 1935 due to seepage slide. The Lyman Dam Arizona, failed in 1915 due to piping. The Hornell Dam, New York, failed soon after construction in 1912 due to seepage. Appendix A contains Middlebrook's comprehensive list of earth dam failures resulting from seepage and/or piping.

are less than those which tend to cause it, the soil particles are washed and piping commences. The resisting forces depend on the cohesion, the interlocking effect and the weight of soil particles, as well as on the action of the downstream filter if any. The erosion starts at the point where the seepage water discharges, and works toward the reservoir. Gradually the seepage channel or "pipe" enlarges until in the final stage complete failure occurs. Terzaghi and Peck (1948a) came up with the concept that piping tends to begin along the contacts between rigid structures and their foundations. The shortest path that a particle of water could take in flowing under a structure was called the line of creep. Terzaghi (1929) demonstrated by means of flow nets that minor geological details can have a major influence on actual seepage patterns in nonuniform foundations. He also restated the physical causes of piping as follows:

> The fundamental requirement is that the upward pull exerted by the flowing water overcomes at some point on the bottom of the tailrace the downward pull exerted by the force of gravity. As soon as this event occurs the dam is lost, no matter where it takes place.

Any uncontrolled seepage errupting in the form of springs in the natural ground downstream from the core of a dam is potentially dangerous. The greater the quantity and velocity of the discharging underseepage the greater the piping potential. The danger is more so if the natural surface soils are fine cohesionless sand and silts. It decrease as the soil becomes coarser, because well-graded coarse soils tend to form natural graded filters at the discharge points. In addition to piping, underseepage sometimes reduces the margin of safety of the downstream slope against shear slides,

Consequent upon occurrence of underseepage, three serious difficulties capable of leading to total failure of the dam face the engineer. They include:-

(i) Piping, which occurs when water picks up soil particles and moves through unprotected exits, developing unseen channels or pipes through the foundation.

(ii) Heave or slope failures (including sloughing) caused by underseepage forces resulting in uplift pressures.

(iii) Excessive loss of water.

We will discuss these problems one after the other.

Piping

As water seeps through the compacted soil of the natural soil of a foundation, the pressure head is dissipated in overcoming the viscous drag forces which resist the flow through the small soil pores.

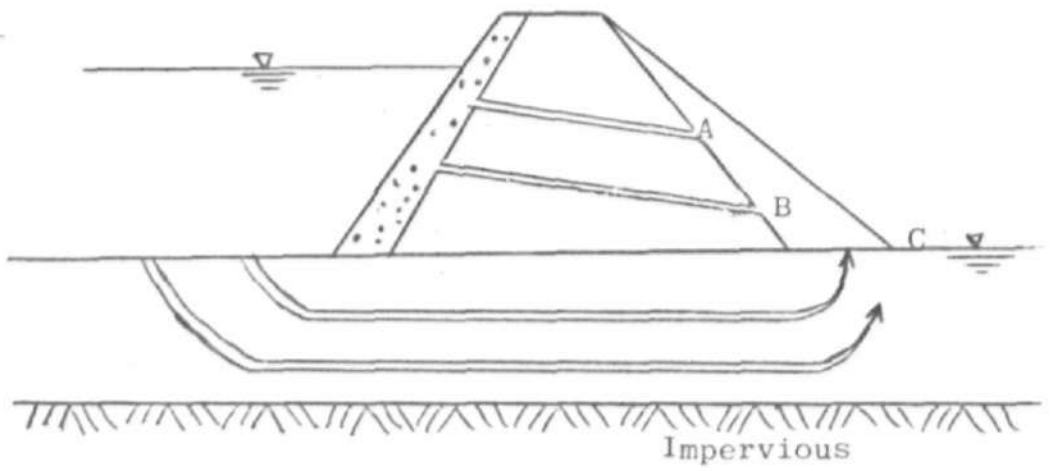


Figure 2.1: Locations within and underneath the dam where piping could occur

Conversely, the seeping water generates errosive forces which tend to pull the soil particles with it in its travel through and under the dam. If the forces resisting erosion

because high pressures develope in the foundation pore water near the downstream toe.

Progressive sloughing (or raveling) is a type of damage which is closely related to piping. It usually occurs on older homogenous dams. The process begins when a small amount of material at the downstream toe erodes and produces a small slump or miniature slide. It leaves a relatively steep face, which becomes saturated by seepage from the reservoir. It then slumps again forming a slightly higher and more unstable face. This raveling process can continue until the remaining portion of the dam is too thin to withstand the water pressure and complete failure occurs suddenly as the reservoir breaks through. Marsal and Pholenz (1972) told about the failure of the Laguna Dam in Mexico from underground piping after 60 years of operation. They maintain that the cause of this failure would have been difficult to anticipate as one of the many unknown channels or "pipes" in the embankment foundation ... may shortcircuit the reservoir in a matter of hours and cause disaster. The dam was built of earth, rolled core.

Uplift Pressure

Nonuniformities in the deposition of soils and vertical holes made by burrowing animals, rotted roots, unfilled drill holes, abandoned water wells, and the like often permit seepage to concentrate and emerge in the form of boils or heave at the landward toe of dams and other hydraulic structures. Piping problems or failures caused by heave can be expected to occur at the downstream side of a hydraulic structure when the uplift pressures of seepage exceed the downward forces

due to the submerged weight of the soil.

Dams are subjected to water pressure, not only on exposed faces but also on their bases and within the masonry itself. These internal pressures produce uplift. Uplift is the upward pressure of water as it seeps or flows through the dam or its foundations. It causes a reduction in the effective weight of the structure above it. The conception generally applied to hydrostatic uplift pressure may be explained by a discussion of the hypothetical conditions illustrated in Figure 2.2. This figure represents a dam with an upstream water depth of h_1 and a tailwater depth of h_2.

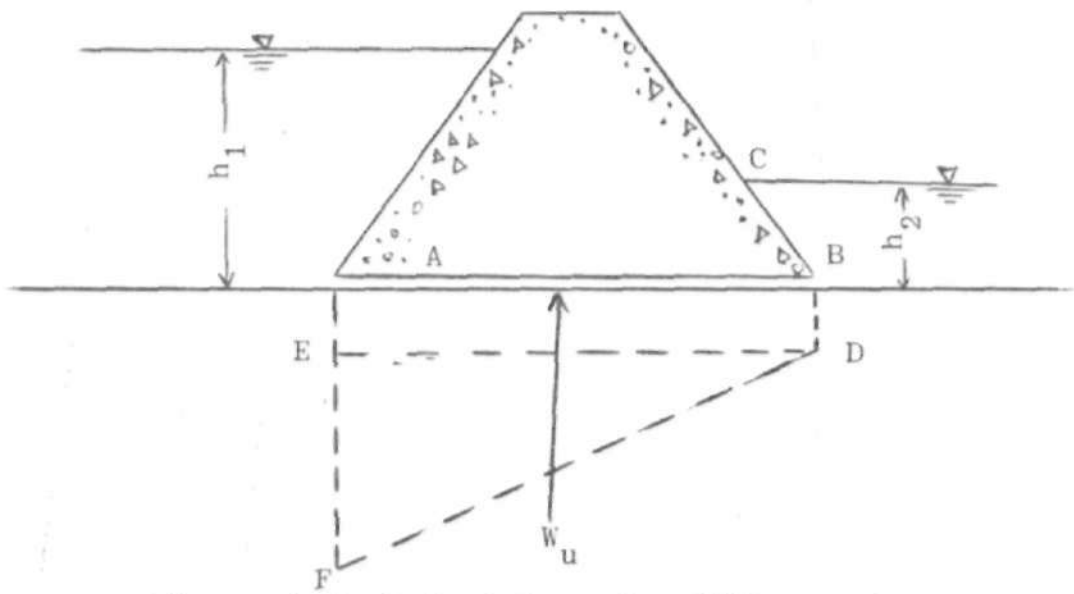


Figure.2.2: Principles of uplift pressures.

Consider the base as slightly raised from the foundation, permitting flow from the upper to the lower pool. Assuming that the width of crack is uniform, that the flow conforms to the laws governing flow in pipes, and neglecting loss at entrance, the pressure intensity will diminish uniformly from w_2h_1 at point A, the heel, to w_2h_2 at point B, the toe. At the toe, it tends to rotate the dam upward about an axis.

Intermediate intensities are represented by the line FD.

If, without lowering the dam, the crack is closed at point B, thus stopping the flow, the pressure becomes constant and equal to w_2h_2, w_2 being the soil unit weight under the dam. If the closure is at point B, the pressure is uniform at w_2h_1. If the obstruction at any of these points is partial, the effects noted will be partially attained.

Actually, the dam is not raised from its foundation but water is forced through the pores and cracks of the masonry and the foundation materials. Water flowing through such pores or cracks follows a similar law of decreasing pressures. However, the cracks or pores are not concentrated along a single line, nor are they always uniformly distributed. Consequently, the actual internal pressures may differ somewhat from those indicated in Figure 2.2.

The uplift pressure in the foundation near the downstream toe of the dam depends primarily on the configuration and permeability of the various subsoil layers. It may be high even if the quantity of seepage is not great. The highest pore pressures develop in the upper elevations of the foundation in the circumstance where there is a relatively impervious horizontal surface layer which prevents the free discharge of free water. If this impervious surface layer is not sufficiently thick, the seeping water usually breaks through a crack or other hole. This may form a sand boil which is a deposition of sand by the upward flow of seepage water around the springs. On the other hand, if it is thick, the underseepage may never be seen at the surface even though dangerously high pressures can be measured with piezometers.

The existence of these forces has been recognized for nearly a century now and has been the subject of much debate. There is perhaps no other factor in the design of masonry dams about which so little is known and about which there are so many differences of opinion (Justin et al, 1964).

Loss of Water

As the action of underseepage continues part of the water in the reservoir is lost. The result is that the dam will no longer be able to serve its intended purpose. It may lead to waste in economy as the dam cannot meet the demand for water supplies either for domestic purposes, agricultural needs, electricity generation requirements, recreational or navigational demands. Apart from loss of valuable water, underseepage may also cause the loss of suspended solids and dissolved matter in that process. On the other hand, if the water is rich in minerals like sodium, seepage tends to flocculate soil having high alumina with the result that the permeability is reduced. Conversely, fresh water may deflocculate clay soils and render them more permeable. This situation for instance led to the failure of dams in Australia.

2.3 Flow of Underseepage

Underseepage may be defined as the flow of water through soil masses of the dam foundation owing to the action of gravitational forces. Except under the most favourable circumstances, precise quantitative estimation of the potential flow of groundwater through unconsolidated deposits beneath a dam is very difficult. To obtain reasonable flow patterns

of flow beneath the dam, materials must be carefully investigated by drilling, pressure and pumping tests. Depending on the configuration of the base of the dam and the space distribution of materials of different permeabilities beneath, upstream, and downstream from the dam, flow patterns range from simple to very complex.

In the usual situation both vertical and horizontal anisotropism in natural materials prevent consistent determination of the flow pattern beneath the dam. Fig.2.3 indicates two kinds of underseepage flow. Both are confined flow but with totally different configurations of flow lines (or streamlines) and equipotential lines (dashed). In Fig.2.3c, the flow is bounded upstream by an equipotential surface at the floor of the reservoir where the potential is equal to H, the head exerted by the water in the reservior, and downstream by a surface of zero head.

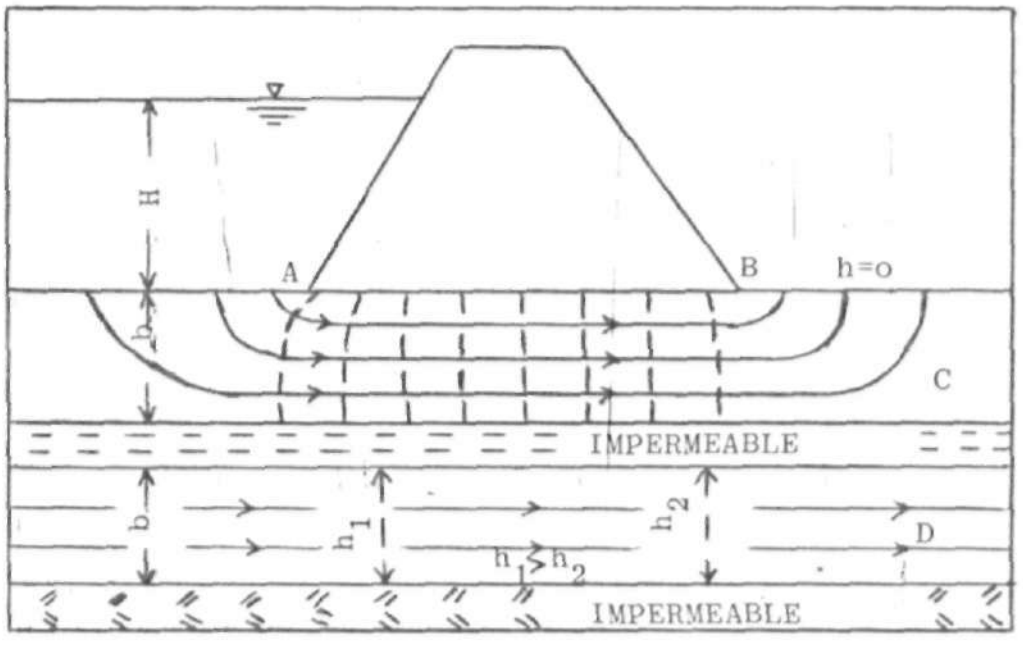


Figure.2.3: Types of groundwater flow beneath a dam.

C: Confined flow above an impermeable layer.

D: Confined flow between parallel impermeable layers.

In Fig.2.3D, the flow is bounded above and below by parallel impermeable layers and the lines of equal head are perpendicular to the confining surfaces and to the streamlines. Generally, since the seeping water is flowing under a gradient toward the downstream toe, the pressure head in the foundation below the dam is always above the tailwater elevation. Therefore if the surface of the ground downstream from the dam is relatively flat, the pressure head measured in a piezometer will nearly always be above the elevation of the ground surface.

An analysis of kind of confined flow suggested by Fig.2.3C is made with the aid of Fig.2.4, and suggests the kinds of problems that are encountered in more complex situations. It is assumed that an average permeability has been ascertained by field measurements. Also, we assume that the materials beneath the dam are perfectly homogeneous in calculations of the order of magnitude of seepage.

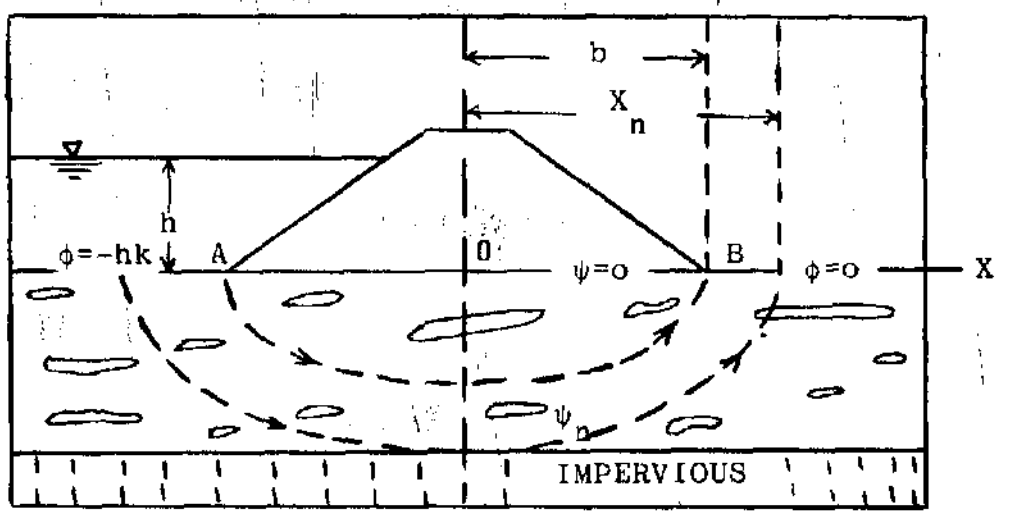


Figure 2.4; Seepage beneath a dam.

The boundary conditions in Fig.2.4 are of the simplest kind. A dam with a flat base rests on a layer of permeable material above an impermeable layer. A horizontal equipotential surface along the floor of the reservoir has a value of ϕ=kh, in which k is the average permeability of the permeable layer, and h is the head in the reservoir. Downstream from the dam the head is equal to zero, and ϕ=o. Flowlines (streamlines) at the base of the dam (ψ=o) and tangent to the impervious layer (ψ_n) have equations such that the total quantity of flow between them (Q) can be calculated from the relationship:

$$Q = (kh/\bar{u}) \cosh^{-1} (X_n/b) \qquad 2.1$$

in which X_n and b are the dimensions indicated in Fig.2.4.

Now, let $X_n/b = u$. Expanding $\cosh^{-1}$, a hyperbolic function, we obtain:

$$Q = (kh/\bar{u})[\log_e 2u-(1/4u^2)-(15/288u^6)-...] \qquad 2.2$$

In Fig.2.3D, the flow is "pipeline" flow in which Darcy's equations in their simplest expression are applicable. Thus, according to Darcy's law;

$$Q = kA(dh/dL) \qquad 2.3$$

in which Q is the quantity of water flowing through a cross-section of area A in a medium with a coefficient of permeability k and a hydraulic gradient dh/dL.

For an aquifer of thickness b and width W the quantity of water flowing through a cross-section of the aquifer is

$$Q = kbW(dh/dL) \qquad 2.4$$

Q is obtained in practice by drill hole measurements of b and W,

field or laboratory measurements of k, and measurements of head in bore holes at points separated by a distance L.

Then,

$$Q = k(bW)(h_1-h_2)/L \qquad 2.5$$

If we designate T = kb as the coefficient of transmissibility, then:

$$Q = TW(h_1-h_2)/L \qquad 2.6$$

2.4 Prevention of Piping and Heave

To prevent piping, water-bearing erodible soils and rocks must never be in direct contact with passageways larger than some of the coarsest soil or rock particles. In nature piping failures are often exhibited by sink holes that form in arid and semiarid lands when fine sand, silt, loess, and clay wash into subtarranean tubes or cracks.

Water that percolates through earth dams and their foundations can carry soil particles that are free to migrate. For example in Fig.2.5, seepage forces at the discharge faces AB and BC tend to cause particles or erodible soil or soft rock

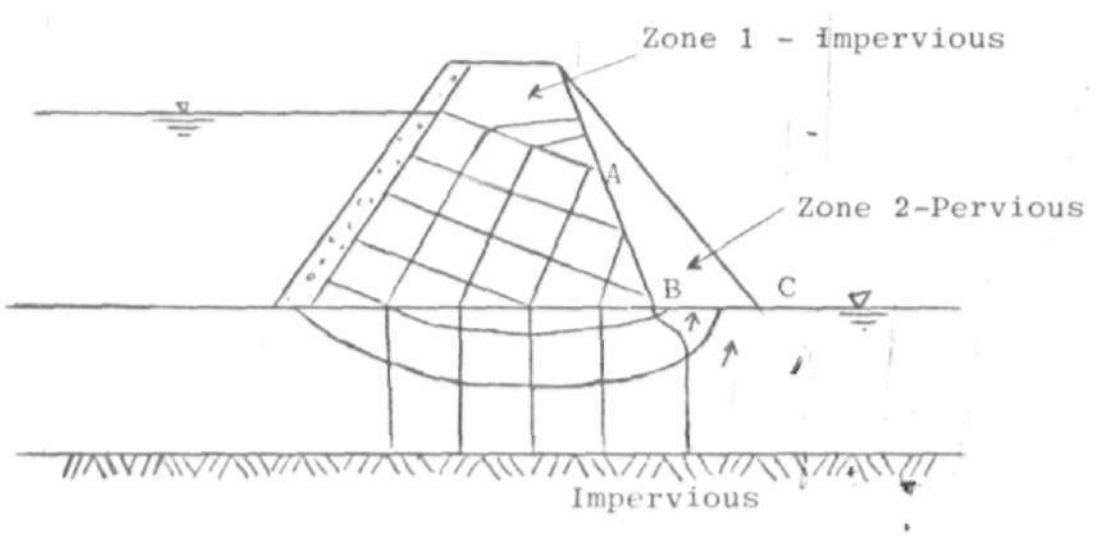


Figure.2.5: Locations within an earth dam, where internal piping must be prevented - faces AB and BC

to move into zone 2 if the seepage forces are great enough and the pore spaces in zone 2 large enough.

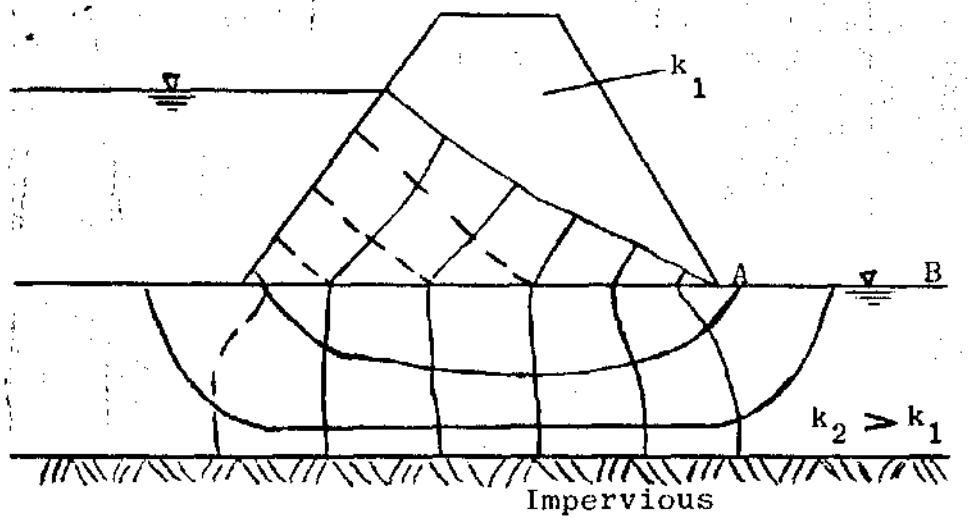


Figure 2.6: Locations downstream of a homogeneous dam where heave may occur - face AB.

Along the unprotected discharge face AB in Fig.2.6, the soil may heave if the gradients are large enough.

Every seepage discharge surface (both internal and external) that could be susceptible to piping or heave must be covered with filters that permit the water to escape freely but hold the soil particles firmly in place. Sound rocks and cemented soils are not susceptible to piping and therefore do not require filters. Nevertheless, soft, weathered sandstones and other erodible rocks and soils must be protected. Flow net studies have proved useful in determining where filters must be provided.

The method described by Terzaghi and Peck (1948a) for determining the factor of safety against piping is illustrated with reference to a row of sheet piles in sand (Figure 2.7)

The principles developed here apply equally to the soil at the landward sides of weirs or dams on permeable earth foundations. First, a flow net is drawn (Fig.2.7a) from which excess hydrostatic pressure on a horizontal plane such as O-x at a depth D can be determined. For a head h_1 on the left side of the sheet pile wall the uplift pressure (excess hydrostatic pressure) can be represented by the ordinates of curve C using line O-x as a reference line.

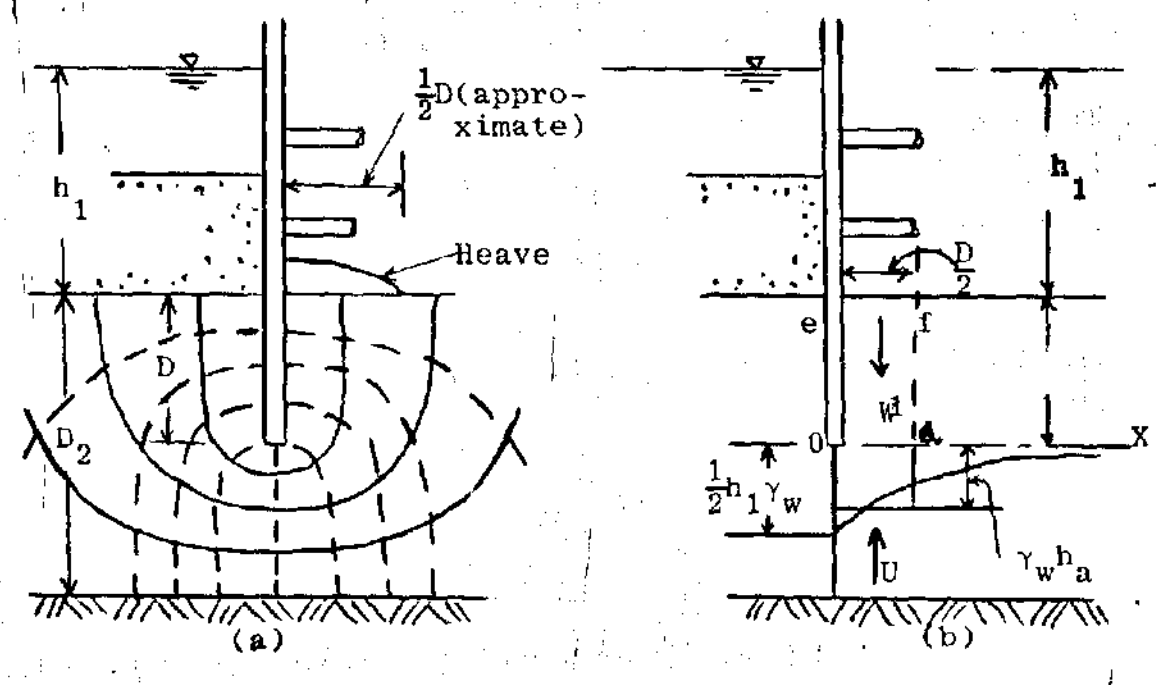


Figure 2.7 Use of flow net to determine factor of safety of row of sheet piles in sand with respect to piping. (After Terzaghi and Peck 1948)

It is shown that the uplift pressure is greatest just to the right of wall; hence the greatest danger exists near the wall. By tests with models Terzaghi found that when the upward forces of seepage on a portion of O-x near the wall become equal to the downward forces exerted by the submerged soil the surface of the sand rises (Fig.2.7a). This heave occurs simultaneously with an expansion of the volume of the

sand, which causes its permeability to increase. Additional seepage causes the sand to boil, which accelerates the flow of water and leads to complete failure. Terzaghi's model tests demonstrated that heave occurs within a distance of about D/2 from the sheet piles. To calculate a factor of safety against failure, forces are determined on the prism efaO (Fig.2.7b) which has a depth D and a width D/2.

The average excess hydrostatic pressure on the base of prism efaO is equal to $\gamma_w h_a$, and the uplift force U is equal to $\gamma_w h_a D/2$. Piping failure occurs when U becomes equal to the submerged weight of the sand with its volume $D^2/2$ times its unit submerged weight γ' or $W' = 1/2D^2\gamma'$.

The factor of safety with respect to piping can therefore be expressed as

$$G_s = \frac{W'}{U} = \frac{D\gamma'}{h_a\gamma_w} \qquad 2.7$$

If it is not economical to drive the sheet piles deep enough to prevent heave, the factor of safety can be increased by placing a weighted filter over prism efaO. If the weight of this filter is W, the total downward force within a distance D/2 of the pile wall is W+W' and the factor of safety is increased to

$$G_s' = \frac{W + W'}{U} \qquad 2.8$$

When a weighted filter is used, G_s is increased because the total stress is increased, whereas the pore pressure remains constant. Normally, the factor of safety should be at least 1.5, to allow for the uncertainties in predicting piezometric heads at critical points.

Criteria for Filter Design

If piping is to be prevented at the discharge surfaces shown in Figures 2.5 and 2.6, the pore spaces in the pervious zone in Fig.2.5 (zone 2) or filters placed at the seepage exits must be too small for soil particles to pass through. If some of the larger soil particles can be held in place, these particles in turn can keep the finer particles from moving.

Bertram (1940) verified that the following criteria suggested by Terzaghi, were conservative.

$$\frac{D_{15}(\text{of filter})}{D_{85}(\text{of soil})} < 4 \text{ or } 5 \qquad 2.9$$

$$\frac{D_{15}\ (\text{of filter})}{D_{15}\ (\text{of soil})} > 4 \text{ or } 5 \qquad 2.10$$

In Eqs. 2.9 and 2.10 the subscripts 15 and 85 refer to the percentage finer by weight than grain size. Equation 2.9 must be satisfied to ensure that filters will be substantially (generally at least 10 to 20 times) more permeable than the soils they protect.

Widespread experience has indicated that when the basic filter criterion expressed by Eq.2.9 is fulfilled in every part of a graded filter, piping cannot occur under even extremely large hydraulic gradients. A graded filter is that which contains two or more layers of different materials, each successive layer being selected on the basis of Eq.2.9.

2.5 Control of Underseepage

As in other engineering works, earth dams, levees and their foundations can be protected from seepage by two fundamental processes: (1) those that keep the water out or reduce the seepage quantities and (2) those that use drainage methods to control the water entering. Methods commonly used under process (1) include:

Cutoff trenches

Grout curtains

Sheet-pile walls and other thin cutoffs

Impermeable upstream blankets

Thin sloping membranes

Methods used under process (2) comprise:

Embankment zonning

Longitudinal drains and blankets

Chimney drains extending upward in embankments

Partial penetrating toe drains

Relief wells.

With regard to control of underseepage, cutoff trenches, grout curtains, sheet-pile walls, upstream blankets and relief wells may be employed and so are treated here. Other methods are outside the scope of this thesis and will not be reviewed.

2.5.1 Cutoff trenches

Vertical cutoffs, usually made of compacted impervious earth trenches normally extend down to an intermediate soil stratum of lower permeability (Fig.2.8). They help to reduce the underseepage discharge by lengthening the passage path. In doing so, it also reduces downstream pore water

pressures, and the moment of uplift forces tending to lift the dam has been reduced. The danger of piping and erosion at the toe of the dam will be reduced or eliminated. Moreover, the longer paths along the stream lines below the

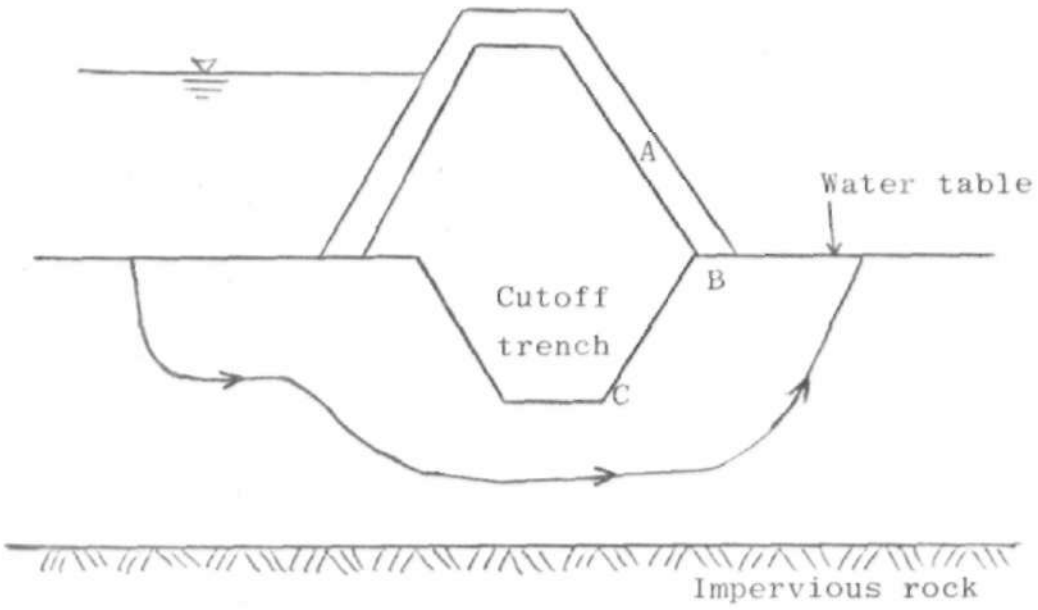


Figure 2.8 Seepage under partial vertical cutoff trench

cutoff causes a notable decrease in the exit velocities downstream in the proximity of the dam and reduces total seepage under the dam.

It is to be noted however that filters must be provided to prevent piping of the soil at faces A-B-C. Theory and model studies by Terzaghi (1929) indicate that it is necessary for a cutoff to penetrate a homogeneous pervious soil foundation at least 95 per cent of the full depth before there is an appreciable underseepage reduction. For this reason, only a complete cutoff should be considered at a site with a homogeneous foundation. In that case, the seepage discharge is nearly zero and the water table falls below B.

On the other hand, a vertical partial seepage barrier is valuable at a site where the average permeability of the foundation soil decreases with depth below the surface or where there is a single continuous impervious layer into which the cutoff can be connected. Where neither partial nor fully penetrating cutoff is warranted, the designer should specify a nominal minimum cutoff trench at a depth of 1.8 to 3.0m for inspection sake. Creager (1917) and Turnbull (1961) presented a graph relating the percentage of depth penetrated by a cutoff trench to the reduction in seepage, as shown below.

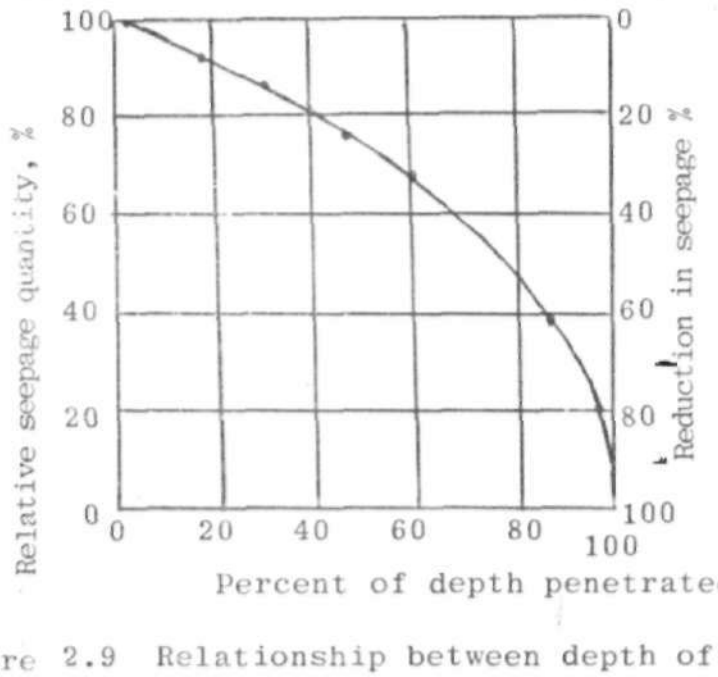


Figure 2.9 Relationship between depth of cutoff and seepage quantity. (After Creager (1917) and Turnbull (1961)).

Trench cutoffs offer a number of advantages because they provide full-scale exploration trenches that expose all soil strata. They permit high quality backfilling operations to be carried out and treatment of exposed bedrock when necessary and the installation of filters and drainage blankets to control seepage and prevent piping into coarse

foundation strata. Also in the process, a large mass of foundation soil is removed and replaced with stronger material, which often helps to stabilize them. Moreover, in zoned dams, it helps to reduce the height of the seepage line in the downstream half of the dam and thereby increase the stability of the downstream slope. In addition, large exit gradients at the toe are curtailed (Sherard, 1963).

2.5.2 Grout Curtains

Grout is a liquid, either of a uniform chemical substance or an aqueous suspension of solids that is injected into rocks or unconsolidated materials through specially drilled boreholes for tightening dam foundations and to reduce or eliminate underseepage. Grouting materials may be Portland cement-based slurries, chemical grouting solutions or organic resins, for example, epoxy resins (polymers).

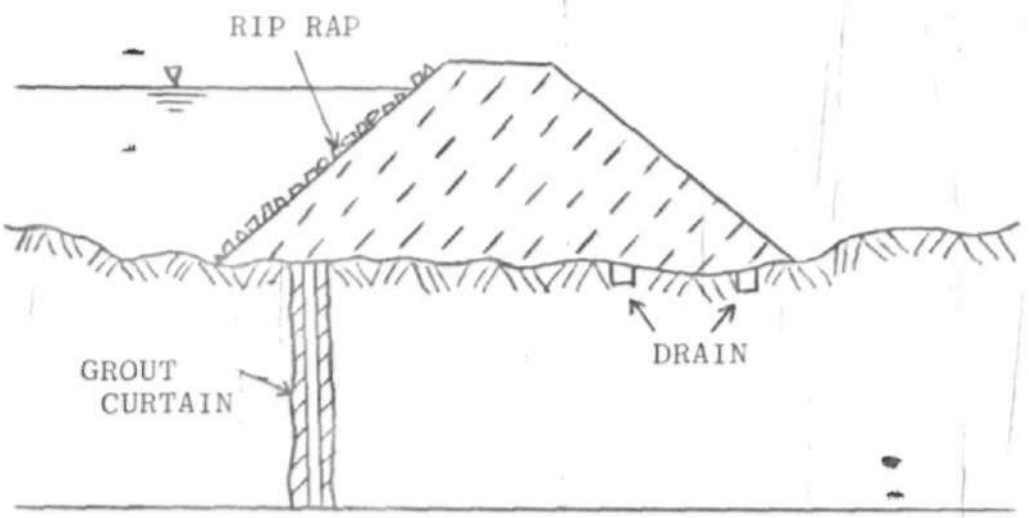


Figure 2.10 Foundation treatment with grout curtain in an earth dam.

In dam foundations three kinds of grouting programmes are identified. (i) comparatively shallow systematic "blanket" or "consolidation" grouting over critical portions of

foundations, (ii) "curtain" grouting from a gallery or concrete "grout cap" along a specified location to produce deep impermeable barrier to subsurface groundwater seepage, and (iii) "off-pattern", special purpose grouting to improve strength and to overcome problems created by groundwater circulation in zones identified by geotechnical studies.

Grouting is not entirely foolproof because cement grout cannot be forced into pores smaller than about 1.0mm and it is not always possible to seal all important joints. Though chemical grouts can be very effective in sealing small rock joints, they cannot always be depended on to control seepage through soils, because they do not always penetrate soils or fine sands sufficiently even with a fluidity approaching that of water. Grout curtains may substantially reduce seepage quantities, but their effect on hydrostatic pressures is often disappointingly poor(Casagrande, 1961).

2.5.3 Sheet Pile Walls

Interlocking steel sheet piling has been used extensively for constructing thin cutoffs through deep alluvial materials as seepage barrier. Because it is expensive and due to possible considerable leakage through the interlocks between the individual piling, it is chosen much less frequently today. However, in some circumstances, where the alternative cutoff methods are expensive or time-consuming, it can provide the best underseepage barrier (Lane et al, 1961). The leakage (and pressure head loss) of water flowing through a steel sheet pile wall depends primarily on the average width of the openings in the interlocking connections.

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As regards efficiency of the sheet pile, Casagrande (1961) demonstrates that a sheet membrane which has 1.58mm slits spaced every 1.52m with an open-space ratio of 0.1% would have a theoretical cutoff efficiency of 29%. A major disadvantage of steel sheet piling is the constant possibility that the wall may be damaged by cobbles or boulders during the driving. Often, with no indication of trouble at the surface, the interlocks tear open or the piling curls. There are also no reliable guides to the relationship between the coarseness or density of a soil and the type of steel sheet piling to be used in it.

In conclusion, seepage resistance offered by a sheet pile wall may be considerable under a low dam where the path of seepage is no more than several hundred metres. The sheet pile wall also is more effective in a homogeneous than in a stratified foundation.

2.5.4 Horizontal Upstream Blanket

Frequently, relatively impervious compacted earth blankets are placed upstream of dams to lengthen the seepage path and thereby reduce the exit gradient and the quantity of underseepage discharge. It also permits cutoff to be placed at some distance from the dam, if such a location were feasible or economical - as may often be the case because of the natural slope of the bedrock. If the blanket is very impervious compared to the natural foundation, the reduction in the seepage quantities and pressures at the downstream toe is directly related to the length and thickness of the blanket.

Good practice for a blanket which is relied upon to control the underseepage requires that it be constructed of impervious soil like that of impervious core of the dam. It also has to completely cover the permeable strata exposed to water pressure. At some dams, nominal blankets have been constructed by dumping impervious soil (sometimes waste stripping or other material) in a random manner and compacting by the travel of the hauling equipment. At other sites, a natural surface blanket of impervious material already exists, and it is necessary only to fill the holes or gaps to make a continuous seal. In such cases, at least the upper surface of the natural blanket should be scarified, brought to a good water content, and compacted.

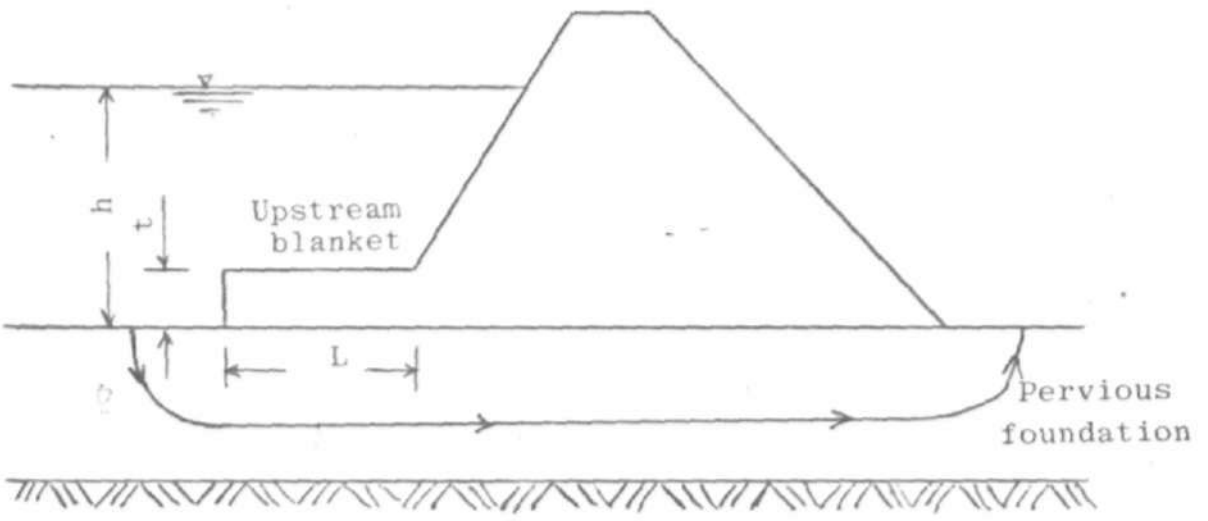


Figure. 2.11 Upstream blanket (connected to the impervious section of the dam).

The necessary thickness, t and length, L depend on the permeability of the blanketing material, the stratification and thickness of the pervious foundation, and the reservior depth, h. Thickness varying from 0.6 to 6m are most frequently used. It has to be stated that even when moderately wide

blankets are used, the line of seepage rises to substantial heights, lowering the stability of the downstream zone. Upstream blanket has proved to be highly beneficial at Vermalion Dam in California (Terzaghi et al, 194).

2.5.5 Relief Wells

The idea of application of relief wells to control underseepage originated from the U.S. Army Corps of Engineers in the late 1930s. Because they are relatively inexpensive and enormously effective in controlling seepage pressures, they are used frequently nowadays. The primary disadvantages of relief wells are: (i) they require inspection and maintenance and they may have to be replaced during the life-time of the dam, (ii) they decrease the average seepage path and increase the quantity of underseepage.

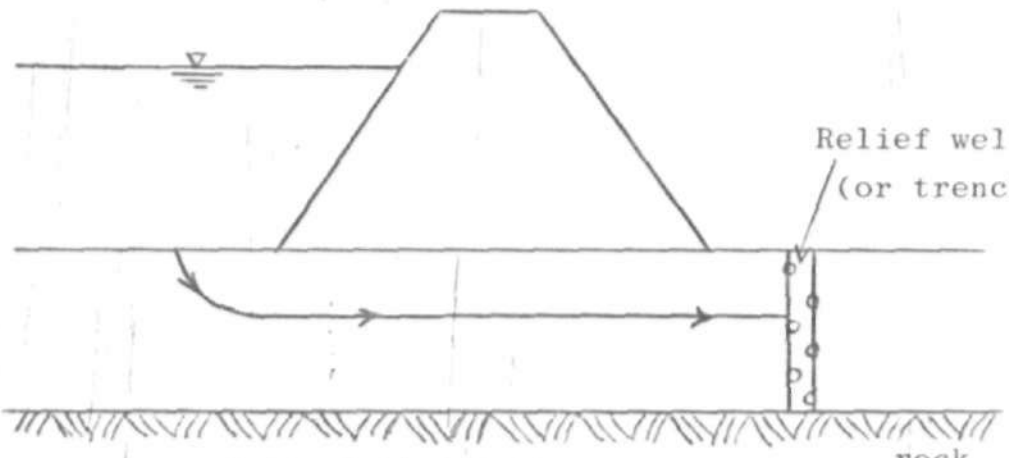


Figure .2.12 Relief Wells

In the immediate vicinity of each well, the pressure head in the seepage water is reduced to a value nearly equal to the elevation of the top of the well (hydrostatic pressure). The wells should be spaced close enough so that the water

pressure between them is not excessive. Practically, they are commonly spaced between 15 to 30m on centres(Middlebrooks et al, 1947). Relief wells have one advantage that intermediate wells can be added later to keep the pressure below any desired value.

They should normally extend through the full depth of the pervious foundation. This is especially important for erratically bedded soil foundations, where the seepage flow pattern cannot be reliably estimated in advance. According to (Turnbull et al, 1961), the effectiveness of wells which penetrate less than 50% of the thickness of the layer is greatly reduced. A relief well ought to have an interior perforated pipe (well screen) with a minimum inside diameter of 15cm, or larger if heavy flows are anticipated. Such screens must be resistant to corrosion and to deterioration caused by bacteria. Gravel-filled holes are much less effective than open pipes (Middlebrooks et al, 1947).

Seepage water from the wells is usually discharged on the ground surface at the toe of the dam through a horizontal overflow pipe. It is then collected in a lined drainage ditch. In general, relief wells frequently offer the most economical positive control over underseepage (Turnbull et al, 1961a). However, in some cases, deep slurry trenches or other thin cutoffs may offer the most practical economical solution Cedergren (1977).

2.5.6 Concrete Walls

Vertical concrete walls of various types have been installed under a few major earth dams as foundation seepage barriers. When founded on rock over their whole length,

concrete walls are perfectly satisfactory and provide almost complete imperviousness.

One of the last major dams in the United States at which a cutoff of this type was used is the USBR's Tieton Dam completed in 1925. There, a 1.5m wide concrete wall was carried down more than 40m. Difficulty with dewatering and shoring during construction discouraged further use of similar designs.

Concrete cutoff walls are still common in England. The excavation is made in the form of a sheeted and braced trench usually of constant width (about 2m). The trench is backfilled with unreinforced concrete, and the water which enters the excavation through the walls and bottom is pumped from sumps. On several English dams these walls have been put down from 45 to 60m(Cedergren, 1977).

A new form of concrete cutoff wall which shows promise in special cases (where its brittleness would not result in cracking and where a deeper barrier is not needed), is a continuous row of overlapping concrete piers installed with special drilling rigs or other equipment. The method is adapted to the use of machinery, and a continuous barrier can be built relatively economically. A number of different systems for constructing the wall without dewatering the ground have been devised. The most common ones are the Intrusion-Prepakt Company method in the U.S. and the "Feder" process of the construction firm ICOS in Milan, Italy. In the Inrrusion-Prepakt method, the piers are made by mixing Portland cement with the natural foundation soil, or by

pumping a sand-cement grout into the hole from the bottom up as the drill is removed. In the Veder process, the holes are held open with a bentonite mud slurry, and the concrete is placed with a tremie from the bottom of the hole.

2.6 Summary

Generally combinations of methods are used. Two or more methods for the control of seepage are employed in most large dams, particularly when they are built on deep aluvial formations or badly jointed rock foundations.

Piping must be prevented by means of filters designed to standards and specifications. Filters can provide permanent security against damaging actions of seepage and groundwater, when certain fundamental requirements are strictly enforced. To prevent the movement of erodible soils and rocks into or through filters, the pores spaces between the filter particles should be small enough to hold some of the larger particles of the protected materials in place Taylor (1948).

An important safeguard to the safety of a dam or reservoir is a good observation and monitoring programme. Periodic reading of piezometers, settlement monuments, horizontal movement stakes, and seepage quantity measurements are required. In addition, various types of seepage tracers should be used to locate areas causing increase in seepage. (Peter et al, 1970) describe the use of one-hole or two-hole methods to observe seepage flow by radionuclid methods. (Birman et al, 1971) describe the use of thermal monitoring to detect leakage and changes in leakage.

Thermister probes calibrated to 0.1°C were read periodically. (Senturk et al, 1970) say that the monitoring of an instrumented dam revealed a sudden 300% increase in seepage. Samples.of seepage quantities should also be tested for suspended solids and dissolved matter because large amounts of matter lost by leaching may cause the formation of channels or cavities, which if uncorrected can lead to piping failures. All references quoted in article 2.6 are to be found in reference number four.

CHAPTER 3

THEORY OF BLANKET DESIGN

3.1 Basic assumptions

The amount of water seeping under an earth dam, together with the distribution of the water pressure can be estimated by using the theory of flow through porous media. The reliability and accuracy of the results however depend on the degree to which the seepage conditions in the dam resemble those postulated by the theory.

In the development of basic concepts the design engineer should keep in mind the theory's limiting basic and simplified assumptions.

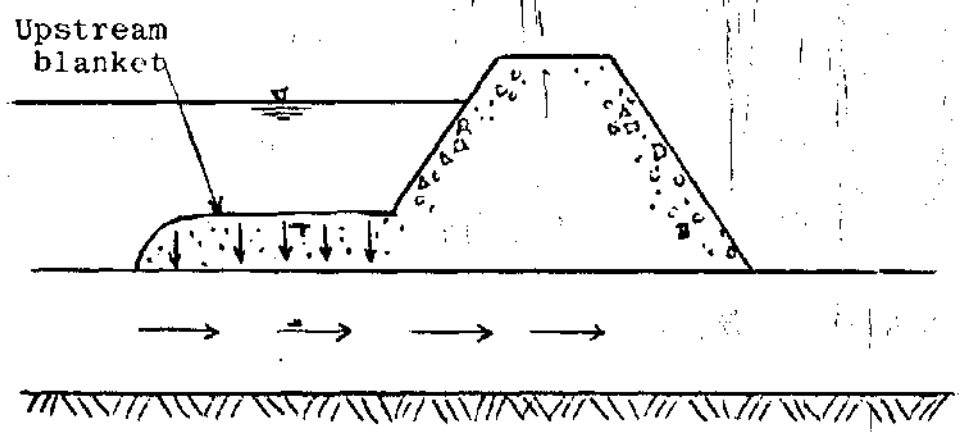


Figure 3.1 Principle of upstream blanket.

In a typical blanket design the following assumptions are generally made: (a) blanket is provided only on the upstream side, (b) the uplift pressure on the downstream side is assumed to be zero. Other assumptions equally important upon which the model experiment would be based include the following:-

(i) The pervious foundation is horizontal and the thickness of the pervious material above it is constant.

(ii) The blanket and foundation materials are homogeneous and isotropic so that their coefficients of permeabilities are constant.

(iii) There is only horizontal seepage flow beneath the dam.

(iv) The seepage flow is horizontal in the pervious foundation material.

(v) In the blanket, seepage is vertically downwards.

For practical reasons, cutoffs under the base of the dam should be provided to control exit gradients and seepage. Also, the above assumptions should be regarded as highly restrictive and may not hold at any locations. However, the theory contained herein provides an invaluable guide to the blanket designer. We shall consider five different blanket shapes which include:

(i) rectangular, (ii) triangular and (iii) trapezoidal shaped blankets. Two composite shaped blankets to be experimented with are stepped-rectangular and rectangular-trapezoidal.

3.2 Rectangular blanket

With reference to Fig.3.2 below, let $k_b << k_f$, where k_b and k_f are the coefficients of permeability of the blanket and foundation materials respectively. All other dimensions are as shown.

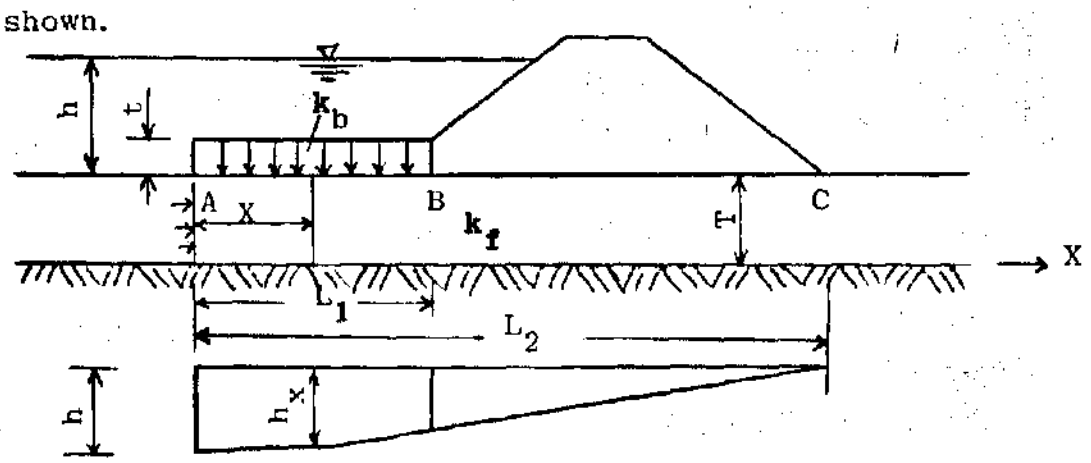


Fig.3.2 Schematic of a rectangular blanket on a dam on pervious foundation and distribution of uplift pressure.

In general form, Darcy's law may be written as

$$\Psi = - k \frac{dh}{dx} \qquad 3.1$$

By the law of conservation of mass and for infinitesimally small element, dh/dx tends to the second derivative of h.

$$\text{Hence } q = - k_f T(d^2h_x/dx^2) \qquad 3.2$$

q = seepage discharge per unit width of dam.

Through the blanket, the seepage discharge is given by

$$q = k_b(h-h_x)/t \qquad 3.3$$

Hence the equation of continuity of seepage flow is written as

$$- k_f T (d^2h_x/dx^2) = k_b(h-h_x)/t, \quad 0<x<L_1 \qquad 3.4$$

Also

$$- k_f T (d^2h_x/dx^2) = 0 \qquad - L_1<x<L_2 \qquad 3.5$$

The necessary boundary conditions are:-

(i) At $X = 0$, $h_x = h$

(ii) At $X = L_1$, h_x is same as in Eq.(3.4) and Eq.(3.5).

(iii) At $X = L_1$, (dh_x/dx) obtained from Eq.(3.4) equals to that obtained from equation 3.5

(iv) At $X = L_2$, $h_x = 0$

Thus the solution of equations 3.4 and 3.5 is

$$(h_x/h) = 1-\sinh(bX)\{b(L_2-L_1)\ \text{osh}(bL_1)+\sinh(bL_1)\}^{-1} \qquad 3.6$$

where

$$b = (k_b/k_f Tt)^{\frac{1}{2}}$$

and

$$(h_x/h) = b(L_2-X)\{b(L_2-L_1) + \tanh(bL_1)\}^{-1} \qquad 3.7$$

Eq.(3.6) is valid for values of X between zero and L_1 while Eq.(3.7) holds for values of X between L_1 and L_2.

When $b \to 0$, the blanket behaves as if it were completely impervious. Thus $\sinh(bX) \to bX$, $\cosh(bL_1) \to 1$ and $\tanh(bL_1) \to bL_1$, so that Eqs.(3.6) and (3.7) become

$$h_x = h(1 - X/L_1)$$
$$h_x = h(1 - X/L_2)$$

which combine generally to

$$h_x = h(1-X/L) \qquad 3.8$$

Eq.(3.8) shows that the distribution of uplift pressure is linear as depicted in Figure 3.1.

Seepage characteristics

Between points A and B (Fig.3.1), the seepage discharge increases uniformly from A to B. It remains constant between B and C beneath the dam in the absence of any infiltration. Therefore the maximum seepage occurs under the dam. Differentiating Eq. (3.7) with respect to X,

$$(dh_x/dX) = -bh\{b(L_2-L_1) + \tanh(bL_1)\}^{-1} \qquad 3.9$$

From Darcy's law,

$$Q_x = -k_f T(dh_x/dX) \qquad 3.10$$

Substituting for (dh_x/dX) in Eq.(3.10) we obtain the maximum seepage discharge beneath the dam

$$Q_m = k_f bhT\{b(L_2-L_1) + \tanh(bL_1)\}^{-1} \qquad 3.11$$

The seepage beneath the dam when there is no blanket is given by

$$Q_o = (k_f h T)/L_d \qquad 3.12$$

where $L_d = L_2 - L_1$

Dividing Eq.(3.11) by Eq.(3.12) gives

$$(Q_m/Q_o) = bL_d\{bL_d + \tanh(bL_1)\}^{-1} \qquad 3.13$$

Optimal design

Suppose for a quantity of the blanket material we wish to optimize its performance. We shall determine the length and thickness of the blanket that will give the minimum seepage discharge and the uplift force therefrom.

Denoting the volume of blanket material per unit width by $\forall$, we have

$$\forall = tL_1 \qquad 3.14$$

Also, suppose

$$b = (k_b L_1/k_f \forall T)^{\frac{1}{2}} = \psi\sqrt{L}_1 \qquad 3.15$$

where b is an abbreviated notation.

$$\psi = (k_b/k_f T\forall)^{\frac{1}{2}}$$

To minimize the seepage discharge, the discharge ratio (Q_m/Q_o) in Eq.(3.13) is to be minimized.

Eq.(3.13) can be rewritten as

$$(Q_m/Q_o) = L_d\psi\sqrt{L}_1\{L_d\psi\sqrt{L}_1 + \tanh(\psi L_1^{1.5})\}^{-1} \qquad 3.16$$

Differentiate Eq.(3.16) with respect to L_1 and set zero, we have

$$\sinh(2\psi L_1^{1.5}) = 3(2\psi L_1^{1.5}) \qquad 3.17$$

solve to get

$$L_1 = 1.263\psi^{-2/3} \qquad 3.18$$

of or $L_1 = 1.263(\Psi/\alpha)^{1/3}$ 3.19

where

$$\alpha = (k_b/k_f T)$$

Substituting for L_1 in Eq.(3.19), we have

$$\frac{\Psi}{t} = 1.263(\Psi/\alpha)^{1/3}$$

such that

$$t = 0.792\ \Psi(\frac{\alpha}{\Psi})^{1/3} \qquad 3.20$$

which can be simplified to give

$$t = 0.792(\alpha\Psi^2)^{1/3} \qquad 3.21$$

Substituting Eq.(3.18) into Eq.(3.16) one gets

$$L_1 = 1.596\ L_d(Q_o/Q_m - 1) \qquad 3.22$$

Note that Eq.(3.22) can be written in the form

$$(Q_m/Q_o) = 1.596\ L_d(L_1 + 1.596\ L_d)^{-1} \qquad 3.23$$

The uplift force per unit width of the dam is given by

$$F = \gamma\int_{L_2}^{L_1}(h_x dX) \qquad 3.24$$

Integrating Eq.(3.24) gives

$$F = 0.5\ \gamma bhL_d^2\{bL_d + \tanh(\psi L_1^{1.5})\}^{-1} \qquad 3.25$$

The reduced uplift force F_R is given by

$$F_R = (2F/\gamma L_d)=L_d\psi\sqrt{L_1}\ \{L_d\psi\ \sqrt{L_1} + \tanh(\psi L_1^{1.5})\}^{-1} \qquad 3.26$$

To minimize F, it is sufficient to minimize F_R which is in fact equal to (Q_m/Q_o), so that the same criterion is obtained as for minimizing the underseepage Gill (1980). Eqs.(3.19) and (3.20) can thus be used for the optimal design of rectangular blankets.

3.3 Triangular blanket

For the analysis of a triangular blanket consider Figure 3.3 given below.

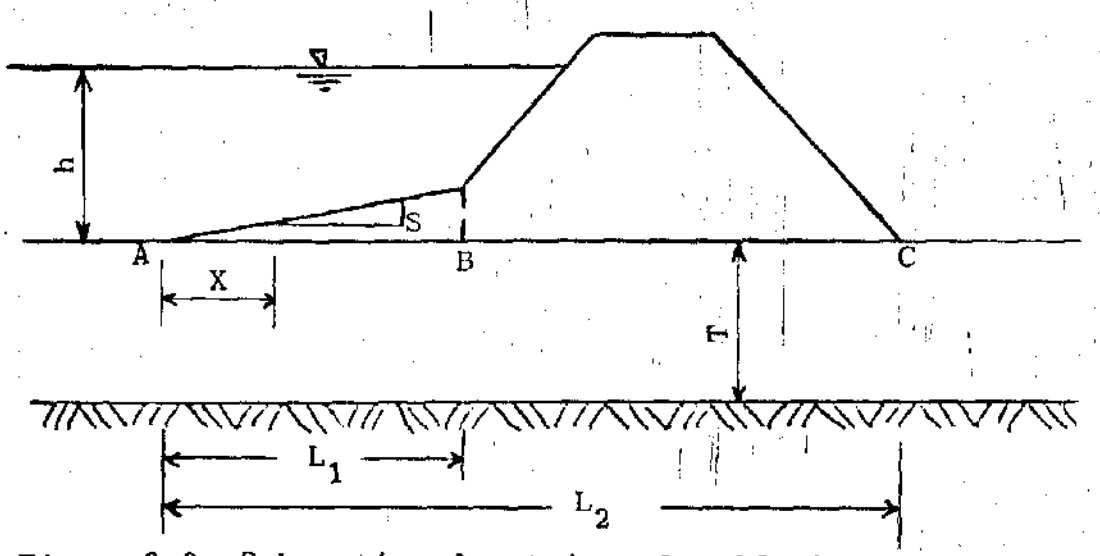


Figure 3.3 Schematic of a triangular blanket.

Eq.(3.4) is modified to read

$$- k_f T(d^2h_x/dX^2) = k_b(h-h_x)/(SX) \quad o<x<L_1 \qquad 3.27$$

where

S = slope of the blanket

Eq.(3.5) is valid as it is. Solution to Eq.(3.5) is directly obtained as

$$h_x = AX + B \quad L_1<x<L_2 \qquad 3.28$$

In Eq.(3.27) let $y = h-h_x$

$$\alpha = k_b/k_f T$$

so that

$$(d^2y/dX^2) = \alpha y/SX \qquad 3.29$$

$$(d^2Z/dX^2) + \frac{1}{X}(dz/dX) - Z(\alpha/SX + 1/4X^2)=o \qquad 3.30$$

Let $\theta = \sqrt{X}$ in Equation 3.30.

Hence,

$$\theta^2(d^2Z/d\theta^2) + \theta dz/d\theta - Z(1 + 4\alpha\theta^2/S)=o \qquad 3.31$$

Eq.(3.31) is the standard form of Bessel's equation with the following solution putting $y = Z\sqrt{X}$

$$y/\sqrt{X} = (h-h_x)/\sqrt{x} = CJ_1\{2(\alpha n/S)^{\frac{1}{2}}\}+DY_1\{2(\alpha X/S)^{\frac{1}{2}}\} \qquad 3.32$$

where

$J_1(\ )$ and $Y_1(\ )$ are the modified Bessel's functions of the first and second kind and of the first order respectively. Using the first boundary condition namely

$h_x = h$ or $y = o$ at $X=o$, it is seen that D must be equal to zero in Eq.(3.32) because $\sqrt{X}\,Y_1(\ )$ goes to infinity as X tends to zero. Eq.(3.32) reduces to

$$y = h-h_x = \sqrt{X}\ CJ_1\{2(\alpha X/S)^{\frac{1}{2}}\} \qquad 3.33$$

With the second boundary condition i.e. at $X = L_1$, h_x is same as in equations 3.4 and 3.5.

$$AL_1 + B = h-\sqrt{L_1}\ CJ_1\{2(\alpha L_1/S)^{\frac{1}{2}}\} \qquad 3.34$$

Using the third boundary condition, i.e. at $X = L_1$. (dh_x/dX) obtained from Eq.(3.4) equals to that obtained from eq.(3.5), Eq.(3.33) is differentiated with respect to X to give

$$(dh_x/dX) = -C(\alpha/S)^{\frac{1}{2}}J_o\{2(\alpha X/S)^{\frac{1}{2}}\} \qquad 3.35$$

Rearranging equation 3.42 gives

$$Q_x = \frac{k_f hTJ_o\{2(\alpha X/S)^{\frac{1}{2}}\}}{J_{o(\tau)}}[L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_{o(\tau)}}]^{-1} \qquad 3.43$$

When $X = L_1$, $Q_x = Q_m$ that is the maximum seepage discharge beneath the dam. Substituting τ for $2(\alpha L_1/S)^{\frac{1}{2}}$ we obtain

$$Q_m = \frac{k_f hT\ J_{o(\tau)}}{J_{o(\tau)}}[L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_{o(\tau)}}]^{-1}$$

Finally,

$$Q_m = k_f hT\ [L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_{o}(\tau)}]^{-1} \qquad 3.44$$

Dividing Q_m by Q_o we have

$$\frac{Q_m}{Q_o} = \frac{k_f hT}{(k_f hT)/L_d}[L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_{o(\tau)}}]^{-1}$$

which is rearranged to give

$$Q_m/Q_o = L_d[L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_{o(\tau)}}]^{-1} \qquad 3.45$$

Assuming that exactly half of the material used in the regular blanket is used in the traingular blanket, we have

$$\Psi = 0.5SL_1^2 \qquad 3.46$$

As defined earlier,

$$\tau = 2(\alpha L_1/S)^{\frac{1}{2}} \qquad 3.47$$

Eliminating S from equations(3.46) and (3.47) gives

$$L_1^3 = \frac{\tau^2\Psi}{2\alpha} \qquad 3.48$$

where

$J_o(\ )$ = modified Bessel's function of the first kind and of the zeroth order. Eq.(3.35) is now substituted into the third boundary condition to give

$$A = -\ C(\alpha/S)^{\frac{1}{2}} J_o\{2(\alpha L_1/S)^{\frac{1}{2}}\} \qquad 3.36$$

Finally the fourth boundary condition i.e. at $X = L_2$, $h_x = o$, gives

$$AL_2 + B = o \qquad 3.37$$

Simultaneous equations (3.34), (3.36) and (3.37) are easily solved to give

$$A = -\ h\ [L_d + \frac{\sqrt{L_1} J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}} J_{o(\tau)}}\ ]^{-1} \qquad 3.38$$

$$B = hL_2[L_d + \frac{\sqrt{L_1} J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}} J_{o(\tau)}}\ ]^{-1} \qquad 3.39$$

$$C = h(\alpha/S)^{-\frac{1}{2}}\ [L_d J_{o(\tau)} + \frac{\sqrt{L_1} J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}}]^{-1} \qquad 3.40$$

where

$$\tau = 2(\alpha L_1/S)^{\frac{1}{2}}$$

Seepage characteristics and optimal design

Substituting for C in equation 3.35

$$Q_x = k_f T(\alpha/S)^{\frac{1}{2}} J_o\{2(\alpha X/S)^{\frac{1}{2}}\} h(\alpha/S)^{-\frac{1}{2}} [L_d J_{o(\tau)} + \frac{\sqrt{L_1} J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}}]^{-1} \qquad 3.41$$

which rearranges to give

$$Q_x = k_f hT J_o\{2(\alpha X/S)^{\frac{1}{2}}\}[L_d J_{o(\tau)} + \frac{\sqrt{L_1} J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}}]^{-1} \qquad 3.42$$

Rearranging equation 3.42 gives

$$Q_x = \frac{k_f hTJ_o\{2(\alpha X/S)^{\frac{1}{2}}\}}{J_{o(\tau)}}\left[L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_{o(\tau)}}\right]^{-1} \qquad 3.43$$

When $X = L_1$, $Q_x = Q_m$ that is the maximum seepage discharge beneath the dam. Substituting τ for $2(\alpha L_1/S)^{\frac{1}{2}}$ we obtain

$$Q_m = \frac{k_f hT\ J_{o(\tau)}}{J_{o(\tau)}}\left[L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_{o(\tau)}}\right]^{-1}$$

Finally,

$$Q_m = k_f hT\left[L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_o(\tau)}\right]^{-1} \qquad 3.44$$

Dividing Q_m by Q_o we have

$$\frac{Q_m}{Q_o} = \frac{k_f hT}{(\overline{k_f hT})/L_d}\left[L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_{o(\tau)}}\right]^{-1}$$

which is rearranged to give

$$Q_m/Q_o = L_d\left[L_d + \frac{\sqrt{L_1}J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}}J_{o(\tau)}}\right]^{-1} \qquad 3.45$$

Assuming that exactly half of the material used in the regular blanket is used in the traingular blanket, we have

$$\Psi = 0.5SL_1^2 \qquad 3.46$$

As defined earlier,

$$\tau = 2(\alpha L_1/S)^{\frac{1}{2}} \qquad 3.47$$

Eliminating S from equations(3.46) and (3.47) gives

$$L_1^3 = \frac{\tau^2\Psi}{2\alpha} \qquad 3.48$$

so that

$$L_1 = (\tau^2 \Psi/2\alpha)^{1/3} \qquad 3.49$$

Therefore

$$\sqrt{L_1} = [(\tau^2 \Psi/2\alpha)^{1/3}]^{\frac{1}{2}}$$

Simplifying the above equation gives

$$\sqrt{L_1} = \tau^{1/3}(2\alpha/\Psi)^{-1/6} \qquad 3.50$$

Substituting for $\sqrt{L_1}$ in Eq.(3.47) and rearranging gives

$$(\alpha/S)^{\frac{1}{2}} = 0.5(2\alpha/\Psi)^{1/6}\tau^{2/3} \qquad 3.51$$

Using Eqs. (3.50) and (3.51) the terms $\sqrt{L_1}$ and $(\alpha/S)^{\frac{1}{2}}$ in Eq.(3.45) can be expressed in terms of τ and Ψ.

The resulting equation is

$$Q_m/Q_o = L_d \left[L_d + \frac{2\tau^{+\frac{1}{3}}(2\alpha/\Psi)^{-\frac{1}{3}} J_{1(\tau)}}{J_{o(\tau)}}\right]^{-1} \qquad 3.52$$

Minimizing Q_m/Q_o gives the following results

$$4/3\tau = \frac{J_{o(\tau)}}{J_{1(\tau)}} - \frac{J_{1(\tau)}}{J_{o(\tau)}} \qquad 3.53$$

Solution of Eq.(3.53) is obtained as

$$\tau = 2.617 = 2(\alpha L_1/S)^{\frac{1}{2}} \qquad 3.54$$

Substituting for S and simplifying yields

$$L_1 = 1.507\ (\Psi/\alpha)^{1/3} \qquad 3.55$$

Comparing equations (3.19) and (3.55) one gets

$$(L_1)_T = 1.193(L_1)_R \qquad 3.56$$

where $(L_1)_T$ = length of the triangular blanket

$(L_1)_R$ = length of the rectangular blanket for the same value of Ψ/α.

Thus for optimal design of a triangular blanket, provide a length which is roughly 19% greater than the corresponding length of an optimal rectangular blanket, Gill (1980).

It can be shown that

$$S = 0.88(\alpha^2/\Psi)^{1/3} \qquad 3.57$$

Using Eq.(3.54) in Eq.(3.45) yields

$$Q_m/Q_o = 1.683\ L_d(L_1 + 1.683L_d)^{-1} \qquad 3.58$$

for which seepage rate can be calculated. Expressing Eq.(3.58) in the form

$$L_1 = 1.683L_d(Q_o/Q_m-1) \qquad 3.59$$

the length of the blanket can be obtained for a specified reduction of seepage rate. Comparing Eq.(3.59) with Eq.(3.22) it is apparent that

$$(L_1)_T = 1.055(L_1)_R \qquad 3.60$$

for the same value of (Q_o/Q_m).

Uplift force

The uplift force per unit width of dam is given by Eq.(3.24). The linearly variable pressure head under the dam is given by Eq.(3.28) for a triangular blanket. For all other shapes of blanket, Eq.(3.28) holds good; except for the constants A and B which will, however, be different for different shapes.

Using Eq.(3.28) for h_x and evaluating the integral in Eq.(3.24) results in

$$F = 0.5\ \gamma A(L_2^2-L_1^2) + \gamma B(L_2-L_1) \qquad 3.61$$

Eliminating B from equation (3.61) using equation (3.37 gives

$$F = - 0.5\ \gamma A(L_2-L_1)^2 \qquad 3.62$$

Observe too that

$$Q_m = - k_f TA \qquad 3.63$$

which is obtained by substituting $dh_x/dX = A$ from Eq.(3.28) into Eq.(3.10). Dividing Eq.(3.63) by Eq.(3.12) yields

$$Q_m/Q_o = - A(L_2-L_1)/h \qquad 3.64$$

Eliminating A from equations (3.62) and (3.64) results in

$$F = \frac{Q_m}{Q_o}\ \frac{\gamma h(L_2-L_1)}{2} \qquad 3.65$$

It should be noted that $\gamma h(L_2-L_1)/2$ is the uplift force per unit length under the dam in the absence of any blanket. Denoting this force by F_o, Eq.(3.65) can be written as

$$Q_m/Q_o = F/F_o \qquad 3.66$$

showing that the percentage reduction in seepage due to the provision of a blanket is the same as the percentage reduction in the uplift force, Gill,(1980).

3.4 Trapezoidal blanket

The general solutions in equations (3.28) and (3.32) hold for the trapezoidal blanket only that the constants A, B, C and D are to be determined for slightly different boundary conditions. The last three boundary conditions remain unchanged, while the first now becomes

$X = L_o$, $h_x = h$ Figure 3.4.

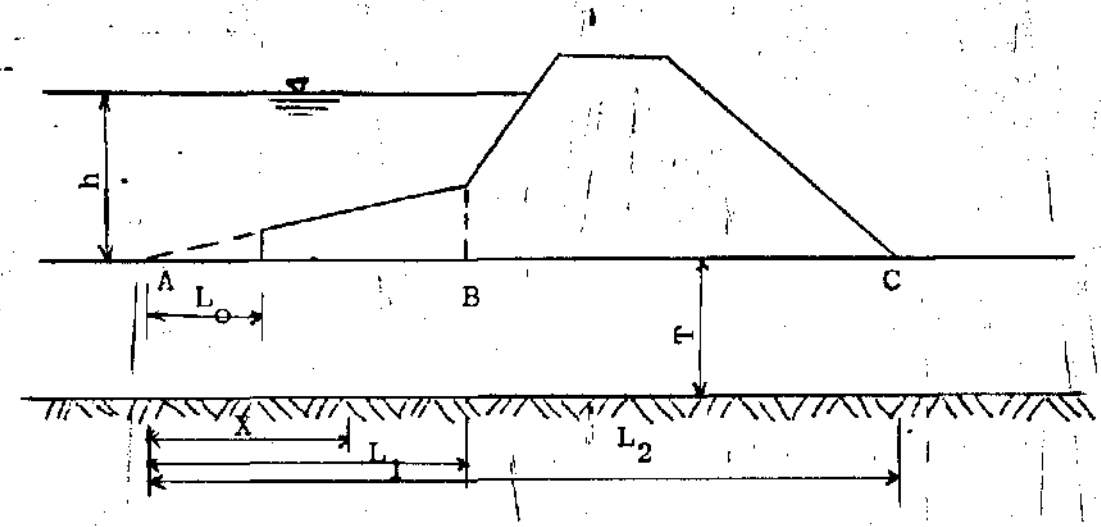


Figure 3.4 Schematic of a trapezoidal blanket

Introducing this boundary condition in equation (3.32) gives

$$D = - \frac{cJ_1\{2(\alpha L_o/S)^{\frac{1}{2}}\}}{Y_1\{2(\alpha L_o/S)\}} \qquad 3.67$$

The use of the second boundary condition yields

$$AL_1 + B = h - \sqrt{L_1}[CJ_1\{2(\alpha L_1/S)^{\frac{1}{2}}\} + DY_1\{2(\alpha L_1/S)^{\frac{1}{2}}\}] \qquad 3.68$$

For the third boundary condition, differentiate equation (3.32) with respect to X to get

$$(dh_x/dX) = -(\alpha/S)^{\frac{1}{2}}[CJ_o\{2(\alpha X/S)^{\frac{1}{2}}\} + DY_o\{2(\alpha X/S)^{\frac{1}{2}}\}] \qquad 3.69$$

The third boundary condition now gives

$$A = - (\alpha/S)^{\frac{1}{2}}[CJ_o\{2(\alpha L_1/S)^{\frac{1}{2}}\} + DY_o\{2(\alpha L_1/S)^{\frac{1}{2}}\}] \qquad 3.70$$

The fourth boundary condition still yields equation (3.37). Solution of equations (3.37), (3.68), (3.69) and (3.70) gives for various constants

$$A = - h\ [L_d + \frac{\sqrt{L_1}\{J_{1(\tau)}Y_{1(\tau_o)} - Y_{1(\tau)}J_{1(\tau_o)}\}}{(\alpha/S)^{\frac{1}{2}}\{J_{o(\tau)}Y_{1(\tau_o)} + J_{1(\tau_o)}Y_{o(\tau)}\}}]^{-1} \qquad 3.71$$

$$B = hL_2[L_d + \frac{\sqrt{L_1}\{J_{1(\tau)}Y_{1(\tau_o)} - Y_{1(\tau)}J_{1(\tau_o)}\}}{(\alpha/S)^{\frac{1}{2}}\{J_{o(\tau)}Y_{1(\tau_o)} + J_{1(\tau_o)}Y_{o(\tau)}\}}]^{-1} \qquad 3.72$$

$$C = h(\alpha/S)^{-\frac{1}{2}}[J_{o(\tau)} + \frac{J_{1(\tau_o)}}{Y_{1(\tau_o)}}]^{-1} \qquad 3.73$$

$$[L_d + \frac{\sqrt{L_1}\{J_{1(\tau)}Y_{1(\tau_o)} - Y_{1(\tau)}J_{1(\tau_o)}\}}{(\alpha/S)^{\frac{1}{2}}\{J_{o(\tau)}Y_{1(\tau_o)} + J_{1(\tau_o)}Y_{o(\tau)}\}}]^{-1}$$

where

$$\tau = 2(\alpha L_1/S)^{\frac{1}{2}} \text{ and } \tau_o = 2(\alpha L_o/S)^{\frac{1}{2}}$$

The constant D can be found using Eq.(3.73) in Eq.(3.67).

Seepage Characteristics and optimal design

The seepage rate given by Eq.(3.9) holds true for the trapezoidal blanket. The maximum seepage rate is given by

$$Q_m/Q_o = L_d[L_d + \frac{\sqrt{L_1}\{J_{1(\tau)}Y_{1(\tau_o)} - Y_{1(\tau)}J_{1(\tau_o)}\}}{(\alpha/S)^{\frac{1}{2}}\{J_{o(\tau)}Y_{1(\tau_o)} + J_{1(\tau_o)}Y_{o(\tau)}\}}]^{-1} \qquad 3.74$$

The term inside the bracket is maximized in order to minimize (Q_m/Q_o).

Hence we maximize

$$\frac{J_{1(\tau)}Y_{1(\tau_o)} - Y_{1(\tau)}J_{1(\tau_o)}}{J_{o(\tau)}Y_{1(\tau_o)} + J_{1(\tau_o)}Y_{o(\tau)}} \qquad 3.75$$

to minimize Q_m/Q_o

Rewrite the expression (3.75) to get——

$$\frac{J_{1(\tau)}}{J_{o(\tau)}} - \frac{\{J_{1(\tau)}/J_{o(\tau)}\}\{J_{1(\tau_o)}Y_{o(\tau)}+Y_{1(\tau)}J_{1(\tau_o)}\}}{\{J_{o(\tau)}Y_{1(\tau_o)} + J_{1(\tau_o)}Y_{o(\tau)}\}} \qquad 3.76$$

The modified Bessel function of the first and second kinds of the zero and first orders are positive, since τ and τ_o have only positive magnitudes. For the expression (3.75) to be maximized, the term following the minus sign in expression (3.76) should be zero. This leads to the condition:

$$J_{1(\tau_o)} = o \qquad 3.77$$

$$J_{1(\tau)}\, Y_{o(\tau)} = - J_{o(\tau)}\, Y_{1(\tau)} \qquad 3.78$$

The condition expressed in Eq.(3.78) cannot satisfy for positive values of τ, and so is disregarded. Eq.(3.77) reduces the trapezoidal blanket to a triangular one which leads to the conclusion that of all the trapezoidal blankets, the triangular blanket is most effective in reducing the seepage quantity, Gill (1980).

3.5 Stepped-rectangular blanket

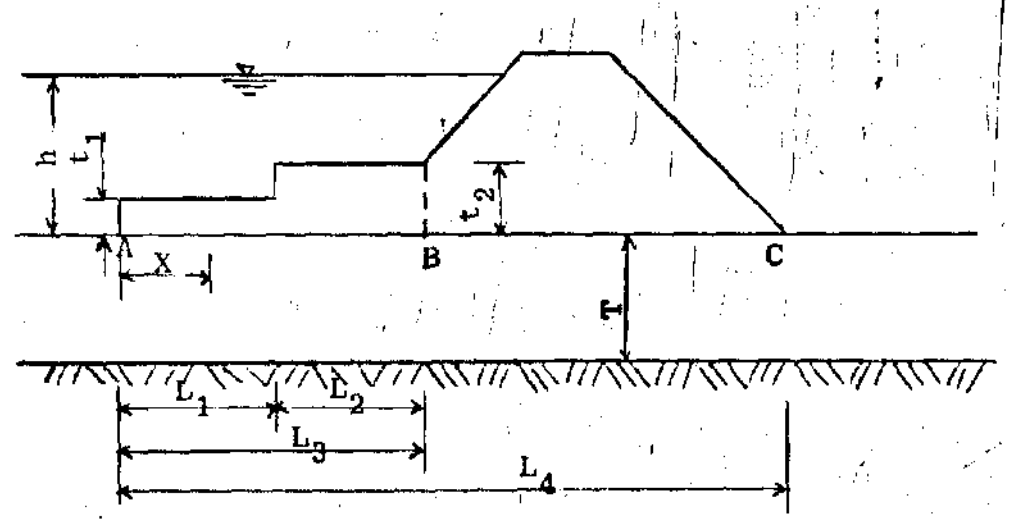


Fig.3.5 Schematic of a stepped-rectangular blanket.

Using the continuity equation of seepage flow, we can write

$$- k_f T(d^2h_x/dX^2) = k_b(h-h_x)/t_1 \qquad o<x<L_1 \qquad 3.79$$

$$- k_f T(d^2h_x/dX^2) = k_b(h-h_x)/t_2 \qquad L_1<x<L_2 \qquad 3.80$$

$$- k_f T(d^2h_x/dX^2) = o \qquad L_3<x<L_4 \qquad 3.81$$

Denote

$b_1 = (k_b/k_f t_1 T)^{\frac{1}{2}}$ and $b_2 = (k_b/k_f t_2 T)^{\frac{1}{2}}$.

The boundary conditions for the rectangular shape hold good with due modifications in the thickness which result into the following equations.

$$(h_x/h) = 1-\sinh(b_1X)\{b_1(L_4-L_3)\cosh(b_1L_1)+\sinh(b_1L_1)\}^{-1} \qquad 3.82$$

$$(h_x/h) = 1-\sinh(b_2X)\{b_2(L_4-L_3)\cosh(b_2L_2)+\sinh(b_2L_2)\}^{-1} \qquad 3.83$$

$$(h_x/h) = (b_1+b_2)(L_4-X)\{(b_1+b_2)(L_4-L_3)+\tanh(b_1L_1)+\tanh(b_2L_2)\}^{-1}. \qquad 3.84$$

Eq.(3.82) is valid for values of X between zero and L_1 while Eq.(3.83) holds for values between L_1 and L_2. Eq.(3.84) is valid for values of X between L_3 and L_4.

Seepage Characteristics

Darcy's law applies such that at any location, the seepage discharge is given by

$$Q_x = - k_f T(dh_x/dX) \qquad 3.85$$

The maximum seepage occurs under the dam for which

$$(dh_x/dX)=-(b_1+b_2)h\{(b_1+b_2)(L_4-L_3)+\tanh(b_1L_1)+\tanh(b_2L_2)\}^{-1} \quad 3.86$$

Thus the maximum discharge of seepage flow is given by

$$Q_m=k_f(b_1+b_2)hT\{(b_1+b_2)(L_4-L_3)+\tanh(b_1L_1)+\tanh(b_2L_2)\}^{-1} \quad 3.87$$

Without any blanket, the seepage under the dam would be

$$Q_o = (k_f hT)/L_d \quad 3.88$$

Dividing equation 3.87 by equation 3.88

$$Q_m/Q_o=(b_1+b_2)L_d\{(b_1+b_2)L_d+\tanh(b_1L_1)+\tanh(b_2L_2)\}^{-1} \quad 3.89$$

Optimal design

Optimizing the length and thickness of the section of the blanket, the length L and thickness t are related by

$$\mathrm{Sinh}(2\psi L_1^{1.5}) = 3(2\psi L_1^{1.5}) \quad 3.90$$

solution of which is

$$L_1 = 1.263\psi^{-2/3}$$

$$\text{or} \qquad L_1 = 1.263(\Psi_1/\alpha)^{1/3}$$

where

$$\Psi_1 = t_1L_1 \text{ and } \alpha=(k_o/k_fT)$$

Therefore

$$L_1 = 1.263\ \{\frac{t_1L_1 \times k_fT}{k_b}\} \quad 3.91$$

From Eq.(3.91) t_1 is obtained as

$$t_1 = 0.792(\alpha\Psi_1^2)^{1/3} \quad 3.92$$

By similar reasoning

$$L_2 = 1.263\{\frac{t_2 L_2 \times k_f T}{k_b}\} \qquad 3.93$$

and

$$t_2 = 0.792(\alpha V_2^2)^{1/3} \qquad 3.94$$

Hence the reduction in seepage can be obtained as

$$Q_m/Q_o = 1.596L_d(L_1+1.596L_d)^{-1} + 1.596L_d(L_2+1.596L_d)^{-1} \qquad 3.95$$

$$= 1.596L_d\{(L_1+1.596L_d)^{-1} + (L_2 + 1.596L_d)^{-1}\} \qquad 3.96$$

3.6 Trapezoidal-rectangular blanket

The shape combines a rectangular section connected to the impervious section of the dam with a trapezoidal section tapering gently upstream, Figure 3.6.

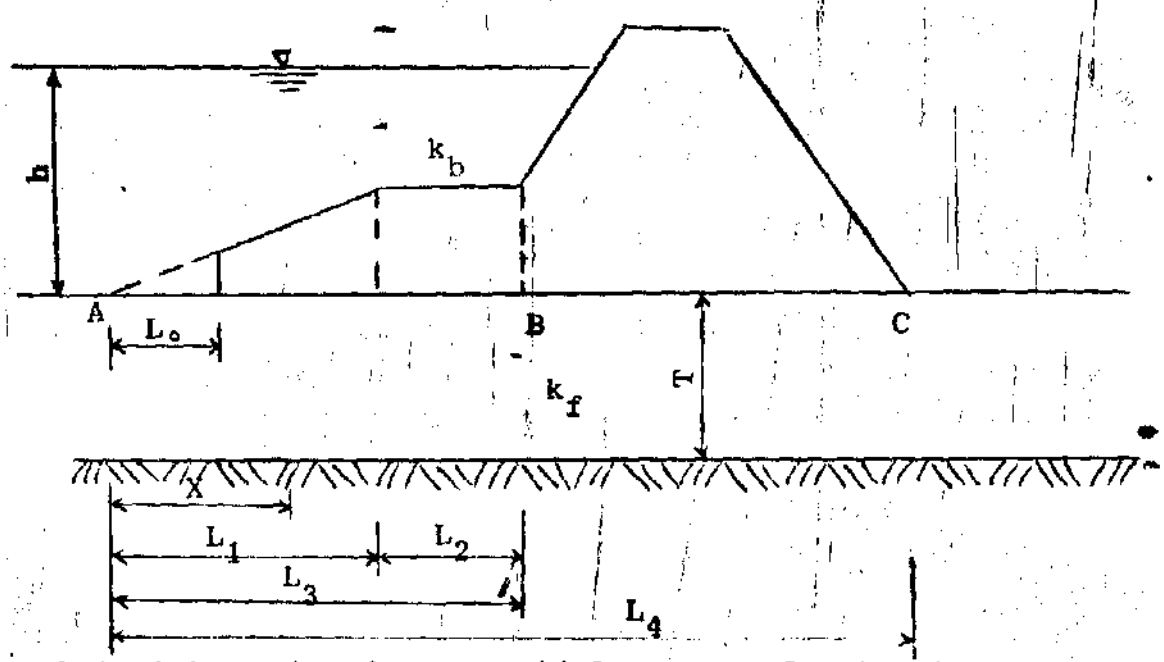


Fig.3.6 Schematic of trapezoidal-rectangular blanket

Equations(3.67) to (3.73) are valid for the trapezoidal section.

$\tau=2(\alpha L_1/S)^{\frac{1}{2}}$ and $\tau_o=2(\alpha L_o/S)^{\frac{1}{2}}$ as before. In part, the maximum seepage is given by

$$(Q_m/Q_o)_T = L_d[L_d + \frac{\sqrt{L_1}\{J_{1(\tau)}Y_{1(\tau_o)}-Y_{1(\tau)}J_{1(\tau_o)}\}}{(\alpha/S)^{\frac{1}{2}}\{J_{o(\tau)}Y_{1(\tau_o)}+J_{1(\tau_o)}Y_{o(\tau)}\}}]^{-1} \quad 3.97$$

The second part, the rectangular section is treated as for the rectangular blanket, where all the equations developed in Article 3.2 hold good.

Hence the expression for partial maximum seepage discharge is written down as

$$Q_m = k_f bhT\{b(L_4-L_3) + \tanh(bL_2)\}^{-1} \quad 3.98$$

and

$$Q_o = (k_f hT)/L_d \quad 3.99$$

where

$L_d = L_4-L_3$ such that

$$(Q_m/Q_o)_R = bL_d\{bL_d + \tanh(bL_2)\}^{-1} \quad 3.100$$

Adding Eq.(3.97) to Eq.(3.100) gives the total reduction in seepage over the entire blanket.

$$Q_m/Q_o=L_d\ [L_d + \frac{\sqrt{L_1}\{J_{1(\tau)}Y_{1(\tau_o)}-Y_{1(\tau)}J_{1(\tau_o)}\}}{(\alpha/S)^{\frac{1}{2}}\{J_{o(\tau)}Y_{1(\tau_o)}+J_{1(\tau_o)}Y_{o(\tau)}\}}]^{-1}$$

$$+ bL_d\{bL_d + \tanh(bL_2)\}^{-1} \quad 3.101$$

It should be noted that in the optimal case, the first term on the right hand side of equation (3.101) reduces to

triangular blanket while the second part gives

$$(Q_m/Q_o)_R = 1.596L_d\ (L_2 + 1.596L_d)^{-1} \qquad 3.102$$

The uplift force is obtained from equation (3.65).

CHAPTER 4

EXPERIMENTATION

4.1 Permeability coefficients

The soil used in the experiments consisted of two types; a clay soil and sand. The sand whose particle size ranged from 0.2 to 1.5mm formed the foundation material. Prior to permeability determination and placement in the tank, it was sieved to determine the size of the grains and later washed to achieve cleanliness. The blanket material was composed of clay with traces of silt.

The coefficients of the permeability of the two types of soil were determined. Permeability as used herein is a measure of the ease with which water flows through soils and rocks. It is of great importance to the water resources engineer when dealing with seepage under dams, land drainage or groundwater lowering. The coefficient of permeability which has been designated by various investigators as Darcy's coefficient, seepage coefficient, effective permeability and hydraulic conductivity is defined as discharge velocity through unit area under unit hydraulic gradient. It is a term in Darcy's law for laminar flow in porous media.

4.1.1 Determination of the permeability of the foundation material

The foundation material used in the experiments consisted of a mixture of medium and coarse sand whose particle size ranged from 0.2 to 1.5mm. Being a cohesionless soil, its coefficient of permeability was determined

using a constant-head permeameter (Plate I)

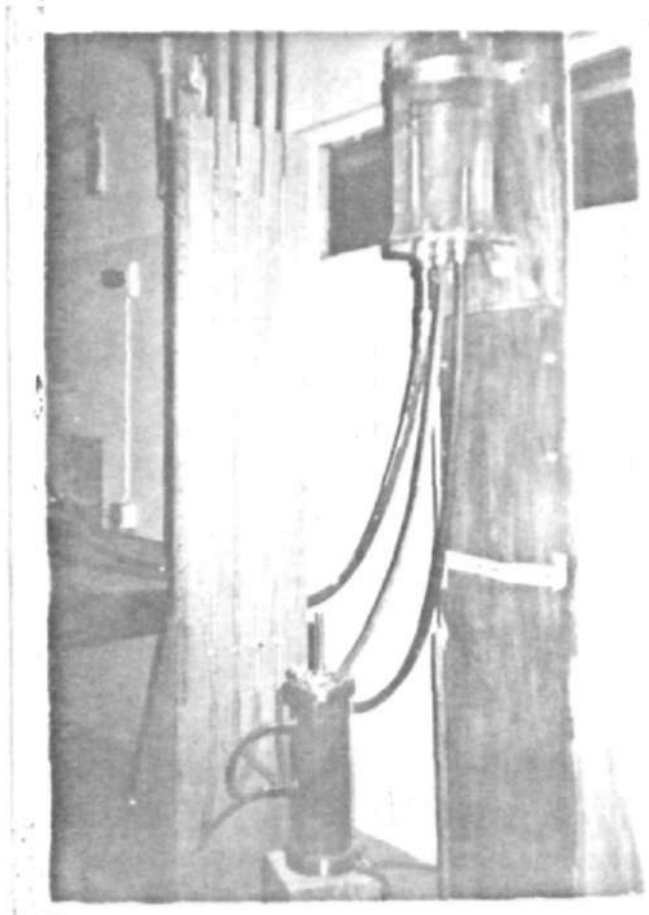


Plate I Constant-head permeameter and the sand specimen

Water under a constant head of pressure was allowed to percolate through a sample contained in a cylinder of cross-sectional area A (Fig.4.1).

When the sample had saturated, the quantity of water q, passing through the sample in time t, was collected in a measuring cylinder. Manometers tapped into the side of the sample cylinder gave the loss of head H, over a length L in Figure 4.1; the hydraulic gradient $i = \frac{H}{L}$. A filter was incorporated above and below the sample to prevent it from washing. From Darcy's law:

$$V = ki \qquad 4.1$$

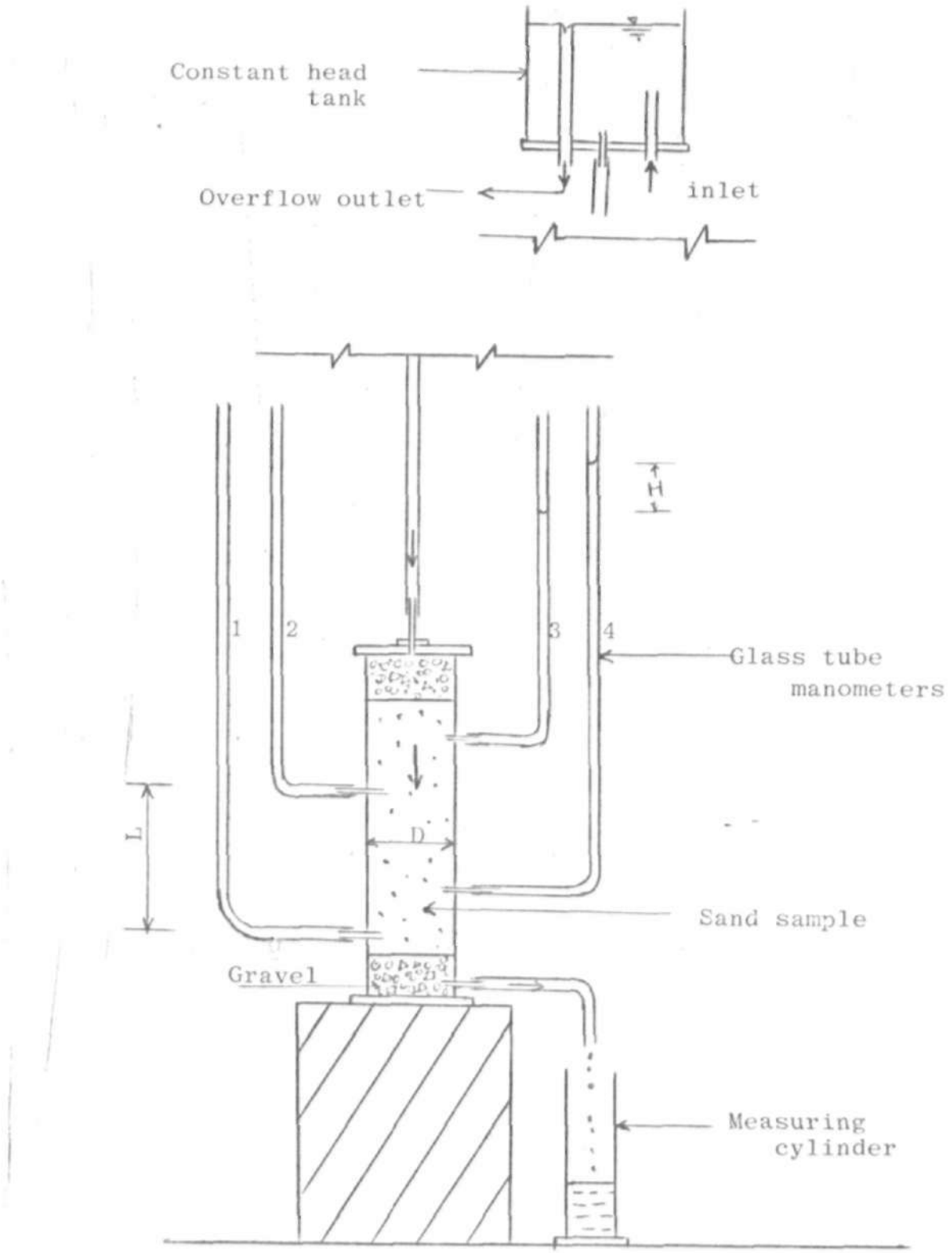


Fig.4.1 Schematic of constant-head permeameter

The discharge per unit time is given by

$$q = Aki \times t \qquad 4.2$$

$$\text{or} \quad q = Ak\frac{H}{l}t \qquad 4.3$$

Rearranging, we have

$$k = \frac{q_o l}{AH} \quad \text{where } q_o = q/t \qquad 4.4$$

Using Eq.(4.4), 10 different values of k were determined. On the average, k was found to be 0.075cm/S. The results are given in Table 5.2.

4.1.2 Determination of the permeability of the blanket material

The blanket material comprised mainly of clay with traces of silt. As the clay is fine-grained, a variable-head permeameter was employed to determine its coefficient of permeability. Using the constant-head permeameter, water would only flow through the fine-grained soil at a much slower rate, so much that a measurable amount of water would not be obtained even after a long time.

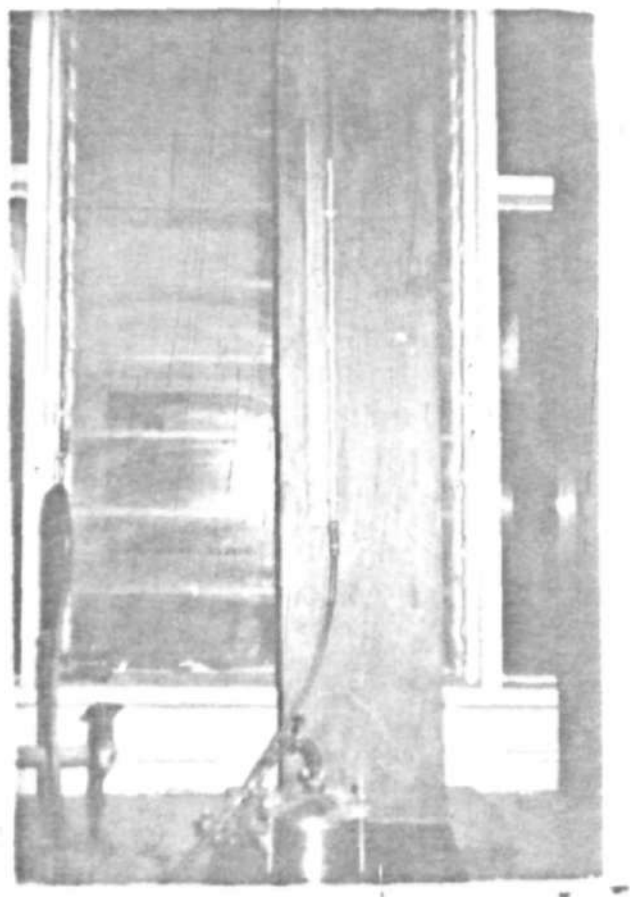


Plate II:The variable-head permeameter containing the clay sample.

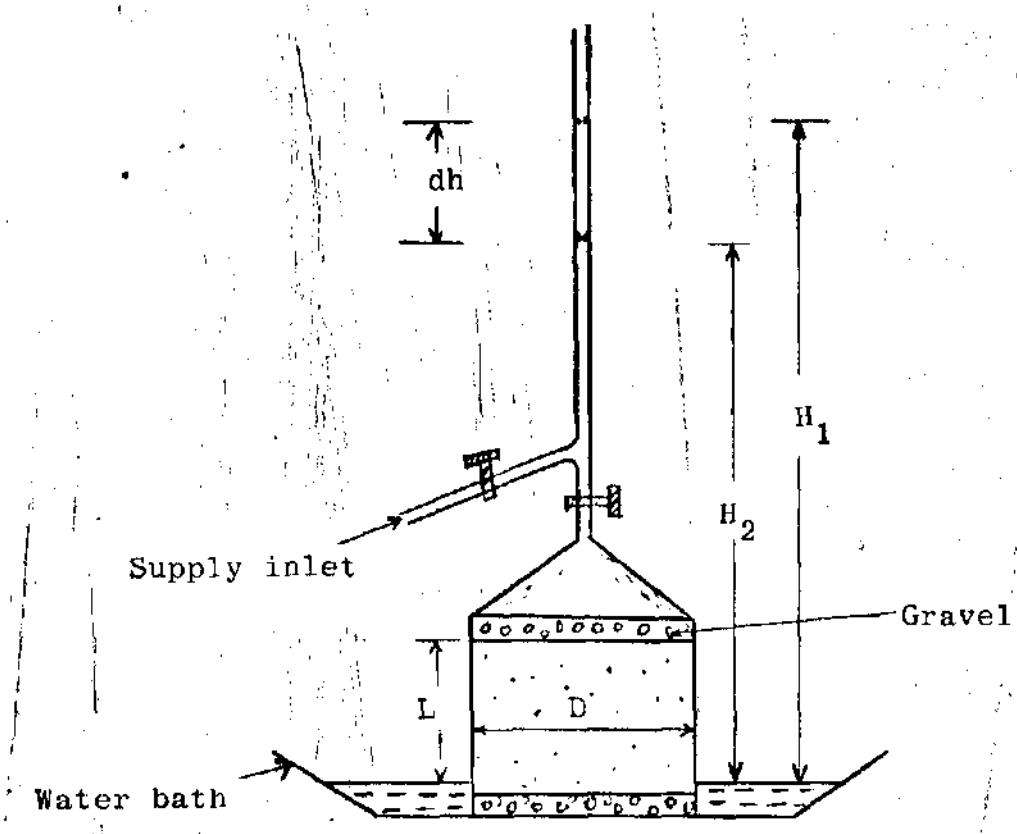


Fig.4.2: Determination of the permeability of clay using a falling-head permeameter.

The sample of length L contained in a 100mm diameter cutting ring were enclosed in a bath containing water. The sample was saturated from both top and bottom. At the bottom was a perforated disk to allow flow of water into the sample from bottom. Also, a wire gauze was incorporated on top of the disk to prevent the washing of the sample. From top, saturation was by means of a branched tube, one arm delivering to the sample and the other to the standpipe of cross-sectional area a. At steady condition of flow, the stop cock on the standpipe was opened to allow flow of water from the pipe through the sample. The time t taken for the head to drop from H_1 to H_2 was taken.

Let h be the height at some time, t. Consider a small interval, dt, and the corresponding change in the level during this time as -dh, (negative as it is a drop in elevation).

The quantity of flow through the sample in time dt is -dhxa and is given by the symbol dQ.

Now $dQ = Akidt$ 4.5

where A is the cross-sectional area of the sample and i the hydraulic gradient (=h/l). Substituting for dQ we obtain

$$-adh = Ak \frac{h}{l} dt \qquad 4.6$$

which rearranges to

$$dt = -\frac{al}{Akh} dh \qquad 4.7$$

Integrating,

$$\int_o^t 1.dt = -\frac{al}{Ak} \int_{H_1}^{H_2} \frac{1}{h} dh \qquad 4.8$$

The solution of Eq.(4.8) is

$$t = \frac{al}{Ak} \log_e(\frac{H_1}{H_2}) \qquad 4.9$$

$$\text{or} \quad k = \frac{al}{At} \log_e(\frac{H_1}{H_2}) \qquad 4.10$$

Expressed in log to base 10, we have

$$k = 2.3 \frac{al}{At} \log_{10}(\frac{H_1}{H_2}) \qquad 4.11$$

Using Eq.(4.11), 10 different values of k were determined. The average of such values gave $k_b = 9.55 \times 10^{-4}$ cm/S.

To ensure that steady conditions of flow had been established, three readings H_1, H_2 and H_3 were taken such

that the time for the head to fall from H_1 to H_2 was the same as from H_2 to H_3. Since in the permeability equation, k, a, A and l are constant, and t had also been made the same, then:

$$\log_{10}\left(\frac{H_1}{H_2}\right) = \log_{10}\left(\frac{H_2}{H_3}\right) \qquad 4.12$$

$$\text{or} \quad \frac{H_1}{H_2} = \frac{H_2}{H_3} \qquad 4.13$$

$$\text{Therefore, } H_2 = \sqrt{H_1 H_3} \qquad 4.14$$

Eq.(4.14) was used to ensure steady conditions of flow before taking readings in a run of the experiment.

4.2 The seepage tank model

The model consisted of four major parts - the seepage tank, the foundation which was made of sand, the blanket which was made of clay and then the dam section. The model was a physical system, normally of smaller size, which could reproduce the physical phenomenon in such a way that the measurements made in the model could be used by the application of appropriate scaling factors to predict accurately the phenomena to be expected in the prototype. The model was sectional in that some features of a dam structure could not be incorporated. However, this would permit the use of a large scale than would otherwise be possible and minimise construction difficulty. The model was operated at a steady flow condition and recirculating system was used so that the model was fully self-contained.

4.2.1 The seepage tank

The walls of the seepage tank were made of 6mm thick glass perspex sheets. It was constructed with all sides vertical, open on top and base impervious. The tank had a total length of 100cm, a depth of 80cm and was 50cm wide. It was fitted with flow devices to regulate and maintain water levels and for discharging the seepage water to the sink and for measurement (Plate III). To prevent warping due to water pressure from the reservoir, the vertical walls were connected on top at two places by means of strips of perspex sheet. Plate III shows the photograph of the tank containing the sand and Fig.4.3 shows the practical dimensions of the tank and the height of the foundation in the model.

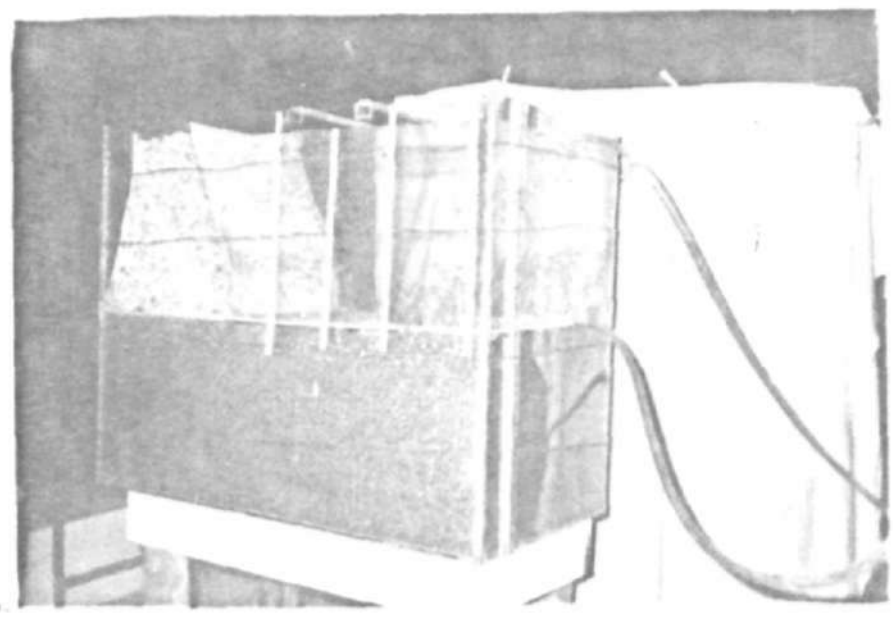


Plate III The seepage tank model containing the sand.

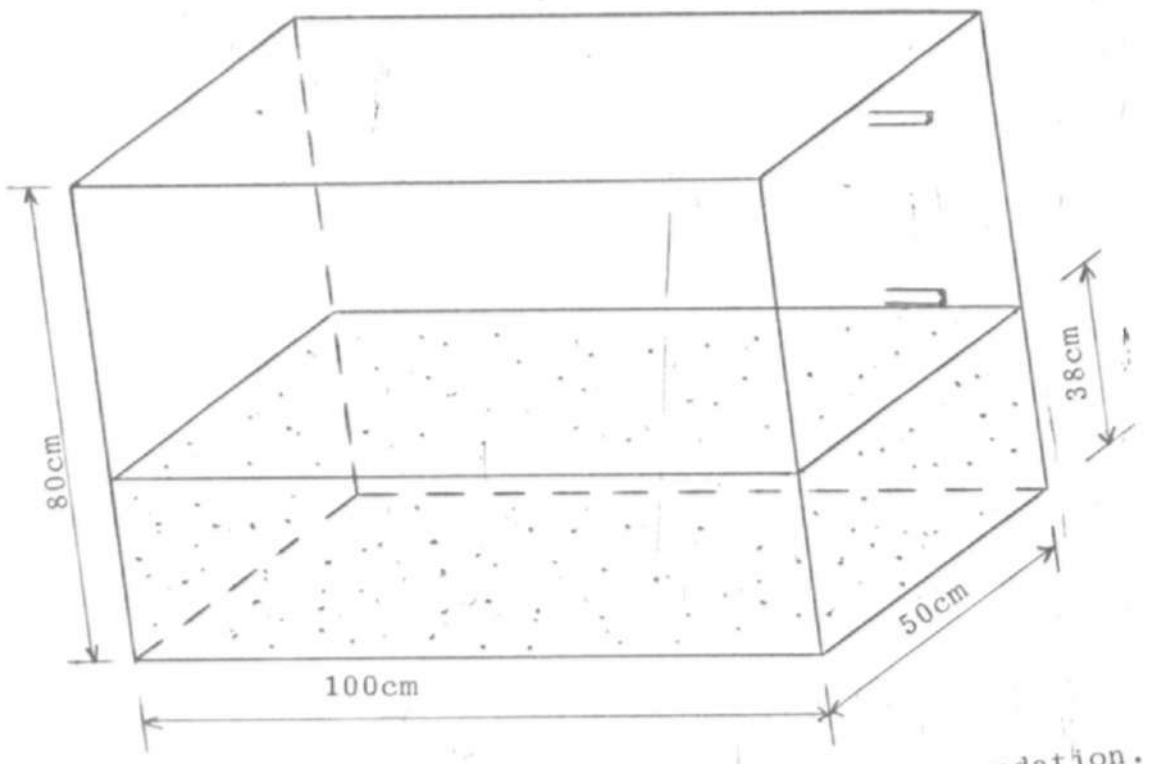


Fig.4.3 Practical dimensions of the tank and the foundation.

4.2.2 The soil materials

The foundation material was carefully packed into the tank and compacted to a height of 38.0cm from the impervious base (Fig.4.3 above). It was covered on top with a perforated 3mm thick plastic perspex sheet to prevent washing by the incoming jet of water especially near the inlet. In addition, prior to covering with perspex sheet, a wire gauze was placed evenly on top of the sand to prevent the washing of the top surface of the foundation. Also, between a distance of 10cm and 60cm from the upstream end toward the downstream end, the cover was perforated using a 3mm drill to admit flow onto the foundation.

The blanket material was gently packed onto the foundation to a height of 10cm at the impervious section of the dam. Depending on the shape of the blanket, the thickness remained constant or thinned out for a length of 50cm toward the upstream end, (Figures 4.4 to 4.8). Each time, the clay was compacted in layers of 2cm using a straight edged piece of wood. Also the uniformity of the thickness was tested by means of a spirit level. To prevent any flow between the blanket and the walls of the tank, little amount of clay was deposited on top of the blanket round the contact point of the blanket and the walls.

4.2.3 The Dam Body

The walls of the dam were made of a 3mm thick impervious plastic perspex sheet to prevent flow of water through the dam. The dam had its edges firmly glued to the tank, using a mixture of perspex chips dissolved in chloroform to prevent any leakage through the dam which would reduce the seepage discharge quantity. The dam base was 40cm wide and 12cm on top. This study does not consider the presence of tailwater, both in the experiments and analyses but where it exists, the downstream blankets should be made much stronger in order to withdstand the unbalanced uplift forces. Moreso, in practice, drainage is usually provided to relieve the uplift pressure. For aesthetic representation, the dam body was filled with Sawdust (plates IV to VIII).

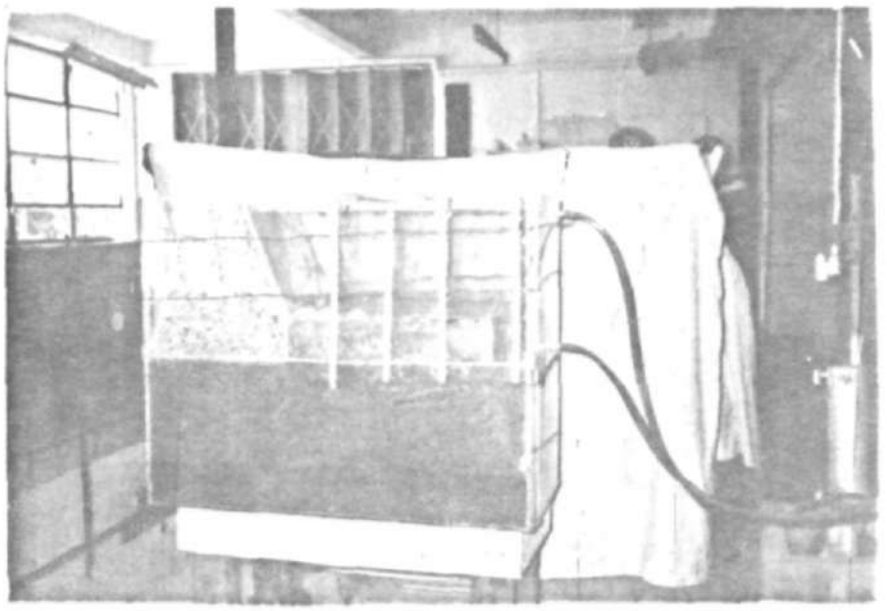


Plate IV: The rectangular blanket.

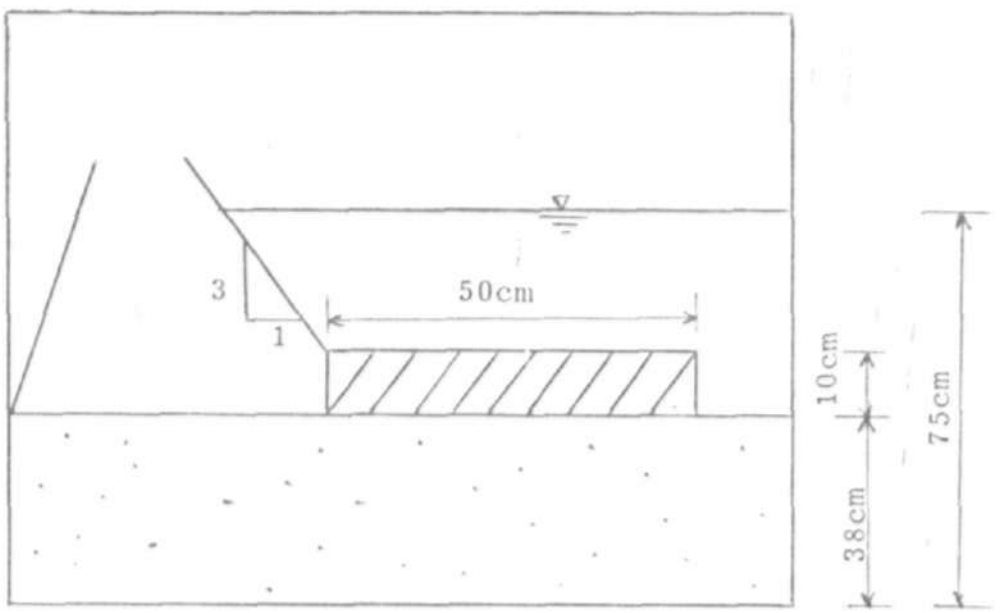


Fig.4.4: Practical dimensions of the rectangular blanket.

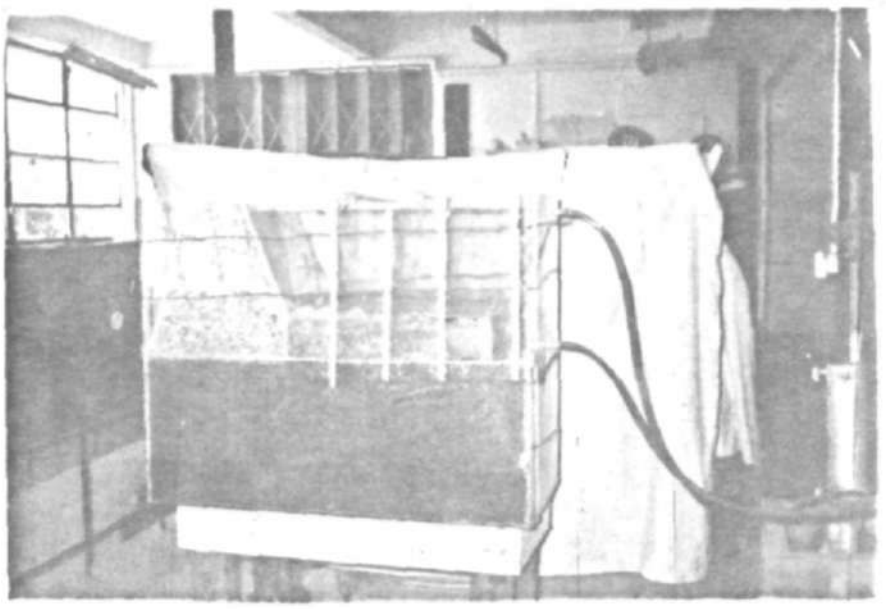


Plate IV: The rectangular blanket.

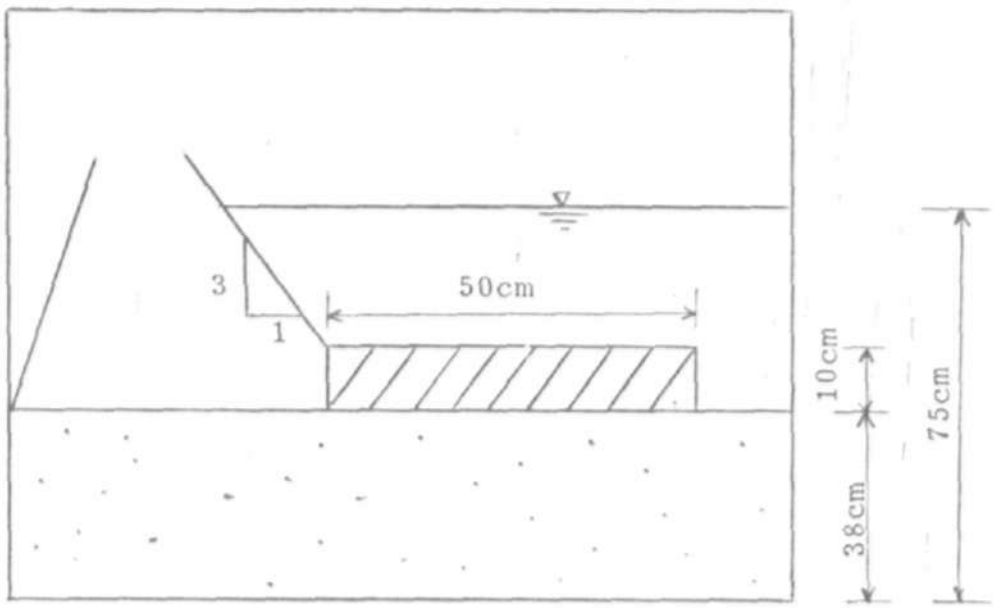


Fig.4.4: Practical dimensions of the rectangular blanket.

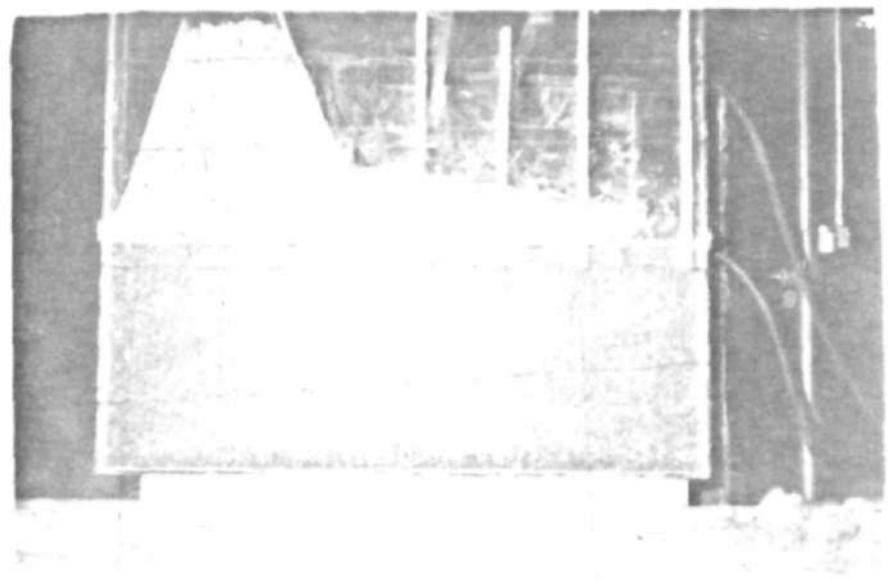


Plate VI: The trapezoidal blanket.

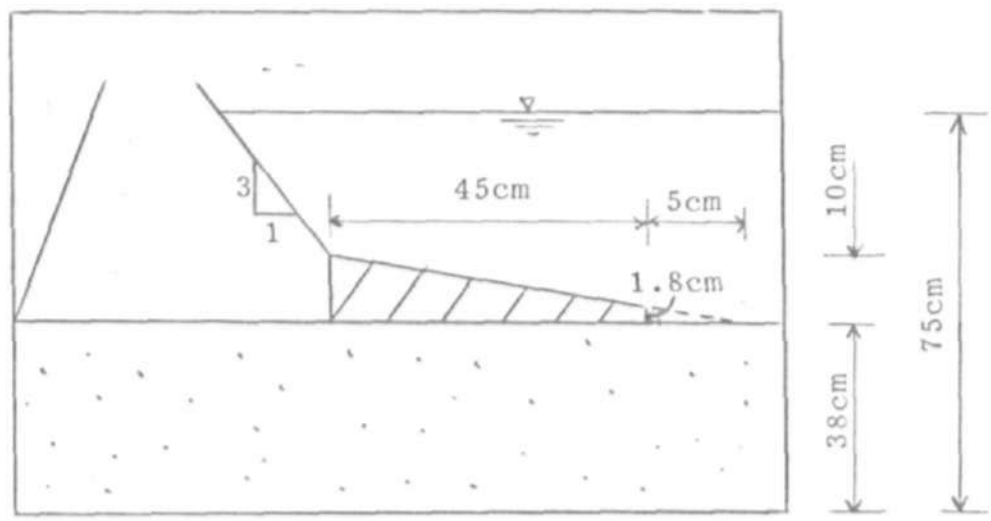


Fig.4.6: Schematic of the trapezoidal blanket with the practical dimensions.

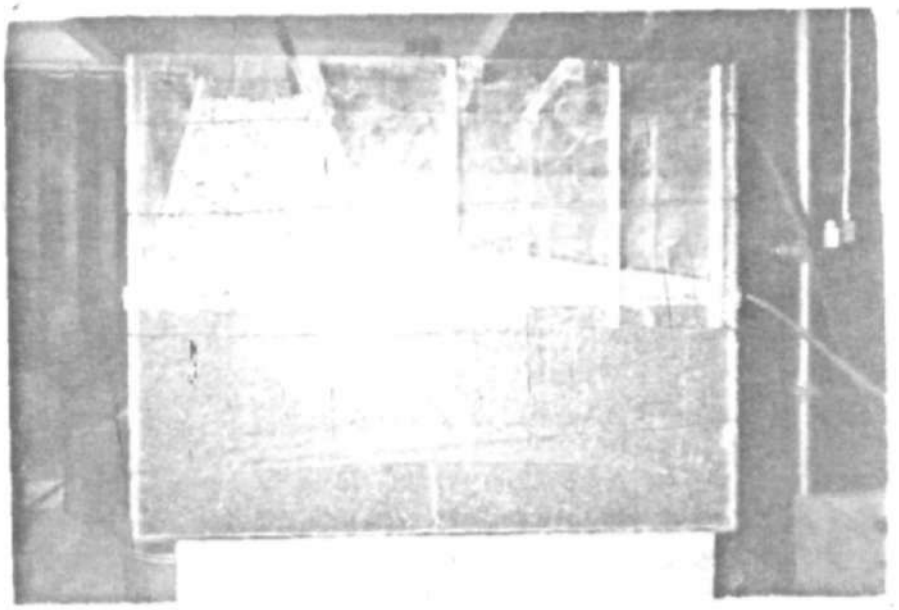


Plate V: The triangular blanket.

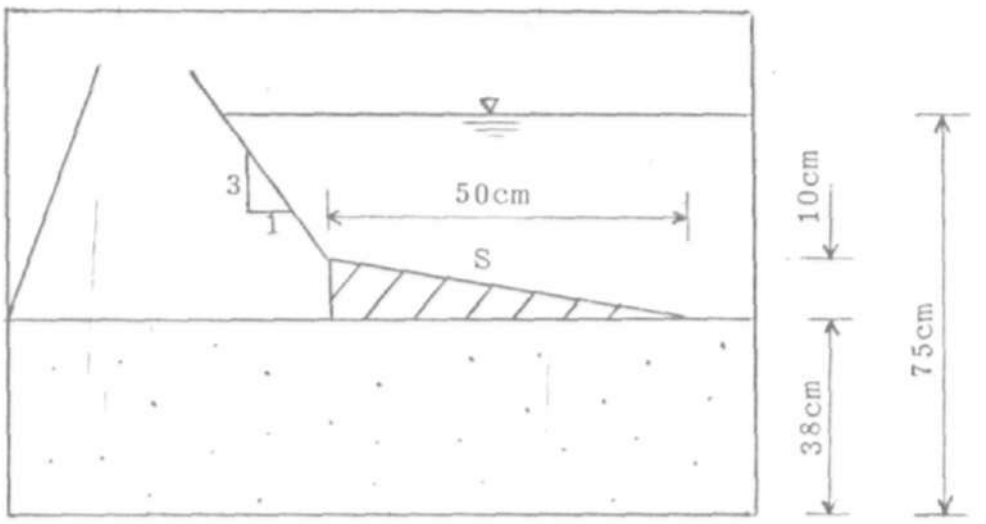


Fig.4.5: Cross-section of the triangular blanket with the applicable dimensions.

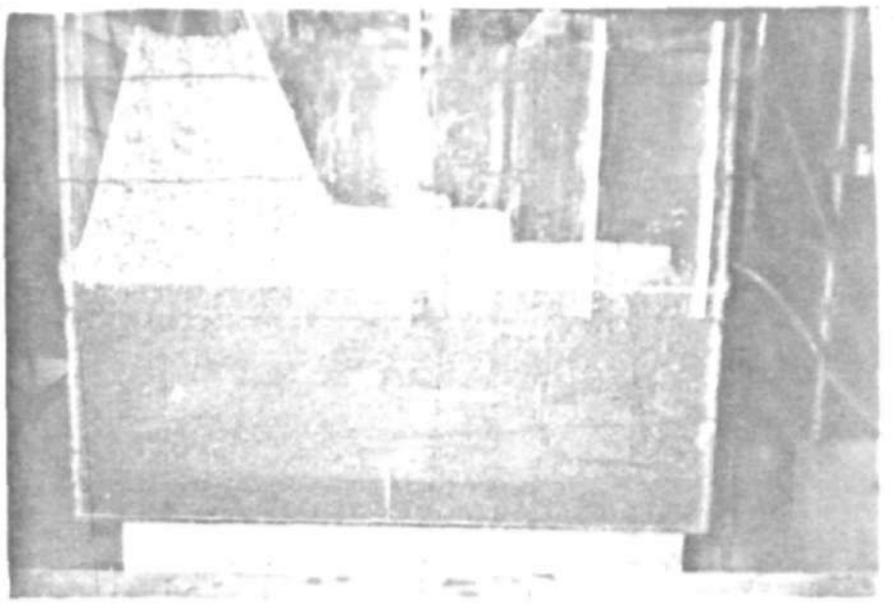


Plate VII: The stepped-rectangular blanket.

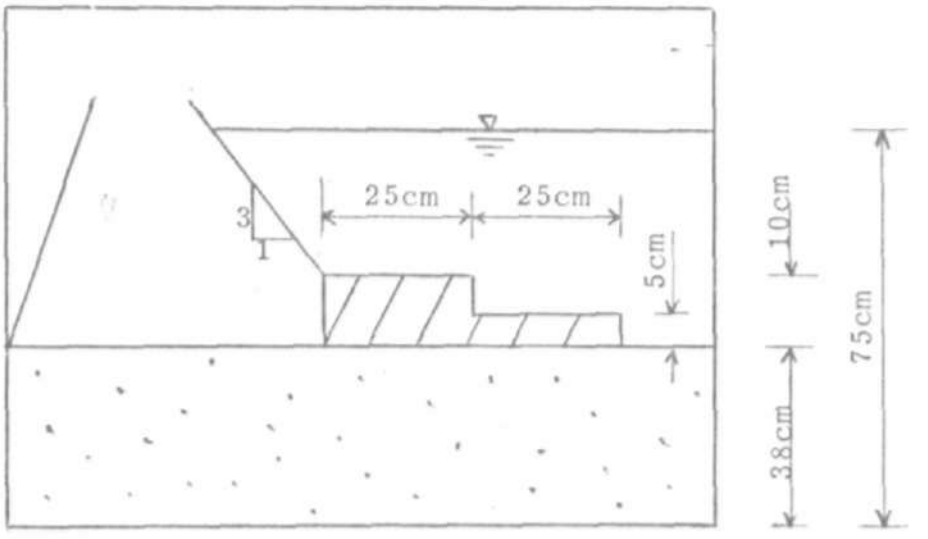


Fig.4.7: The numerical dimensions of the stepped-rectangular blanket.

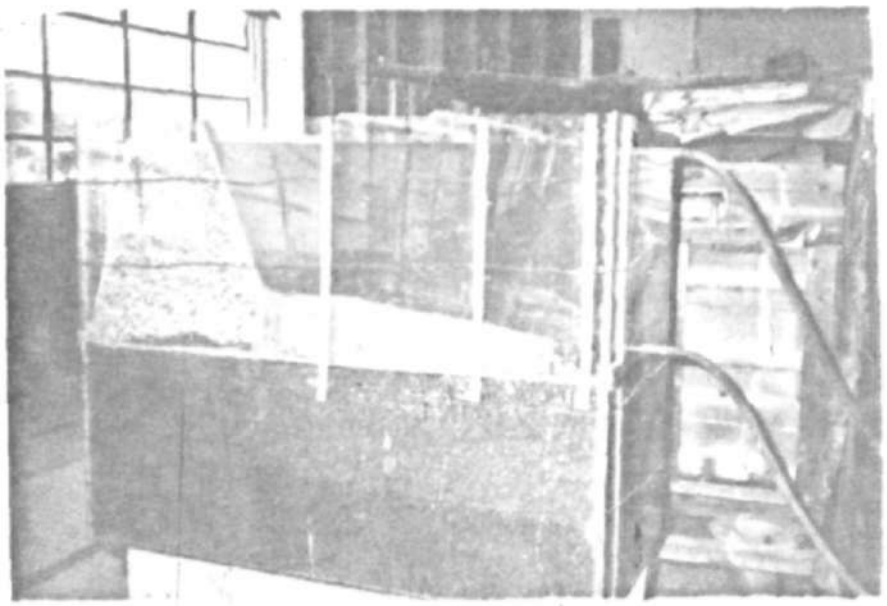


Plate VIII: The rectangular-trapezoidal blanket.

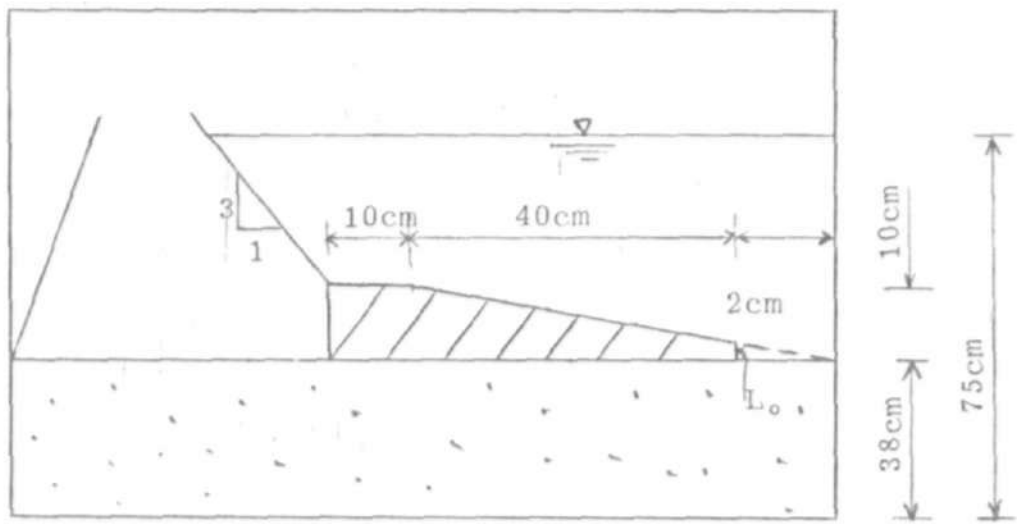


Fig.4.8: The practical dimensions of the rectangular-trapezoidal blanket.

4.3 Measurement of seepage discharge

At the upstream end was the inlet hole made of a 6mm perspex tube connected to a hose pipe through which water was supplied to the system. The inlet hole was drilled 3mm upward from the foundation and discharged water, initially

at a slow rate, onto the perforated cover resting on the foundation. At 10cm from the upstream end toward the downstream side, a 10cm high partition wall was placed. This served to prevent the jet of incoming water from hitting and washing the blanket. For the same purpose the blanket top was covered with a perforated 3mm perspex sheet. The water rose gently up the partition wall and fell over onto the blanket surface. It then moved gradually toward the dam with corresponding increase in the reservoir height until it reached the constant head of 37cm above the foundation.

The water seeped vertically through the blanket and horizontally through the foundation. The discharge emerged through a piece of perspex tube (10mm diameter) drilled at the middle end of the impervious base. The free end of the tube was fitted to a hose pipe which conveyed the underseepage discharge to the sink, where the respective measurements were undertaken. To prevent washing of the finer particles through this hole, a sponge was used to cover it before packing in the sand.

For a given shape of blanket, and when the steady condition of flow had been achieved, the discharge for a period of time t was measured by means of a graduated cylinder and a stopwatch (Plate IX). The amount of volumetric underseepage discharge divided by t gave the underseepage discharge per unit time, Q. For each blanket shape, 10 such readings were taken and averaged. The average Q



Plate IX: Measurement of the underseepage discharge.

divided by 40 gave the mean maximum underseepage discharge per unit time per unit width of dam, Q_m.

Prior to the introduction of any blanket, the maximum underseepage discharge per unit time per unit width, Q_o was established. This quantity formed the means of assessing the performance of a given shape of the blanket.

A tracer solution or dye made of potassium permanganate was used to determine whether or not there was a flow between the blanket and the tank wall or a concentrated flow through the blanket due to cracks or presence of voids. Whenever the result indicated positive, more of the clay was used to block the hole and more time was allowed for steady-state condition of flow to resume before taking readings. The results of the experiments are given in Tables 5.5 through 5.9.

4.4 Precautions

To achieve the desired goals of the thesis, the following precautionary measures were taken right from the collection of the soil materials to the measurement of the underseepage.

In the first instance, all inorganic impurities were hand-picked and the sand sieved to prevent too many fines and undersired grain sizes. The sand was then washed for purpose of fair cleanliness. It was then carefully packed into the seepage tank and evenly compacted to ensure homogeneity and isotropism in the foundation. The clay which was relatively undisturbed was divided into five portions so that one portion was used for only one blanket and then descarded. The merit of this measure was that the permeability of the soil would be retained likewise the natural moisture content as the portions yet to be used were wrapped in cellophane bags.

Readings were commenced only after total steady condition of flow had been established and the clay had become fully saturated. This usually took one week, which would ensure equal flow across the entire top surface of the blanket. During the measurements of seepage discharges, for any one run of the experiment, the time lags were kept at barest minimum to avoid inconsistent readings due to possible fluctuations in the reservoir head.

4.5 Experimental difficulties

Due to low compressibility of dry sand in large volume and the fragile nature of the tank, the sand could not be

subjected to high compaction as it was in the permeameter. The size and continuity of the pore spaces or joints through which the water flowed in the permeameter and the tank could not be kept the same as a result of the inevitable lower compaction in the seepage tank model. Also, the compaction of clay in the model could not be the same as in the permeameter, causing a variation in the predetermined value of the permeability. Moreover, the effects of the prevailing temperature and viscosity of water could introduce variations in the permeabilities.

In summary, the laboratory measurement of permeability has one disadvantage that the test sample was small and probably not representative of average conditions in the aquifer. There is also a possible difficulty in obtaining undisturbed samples which represent the true condition in the aquifer. However, the standard permeameter used has a diameter greater than 40 times the mean particle diameter, which helped to avoid wall effect. To avoid difficulties posed by air bubbles, the water was dcarated, and the medium was saturated before test. The net result of the above variations lies in the belief that we have accurate measurements in our grasp, and can confidently expcet to achieve nearness to the true values.

CHAPTER 5

ANALYSES OF RESULTS

In carrying out the experiments, five different shapes of the blanket were used. These are the rectangular shape, the triangular shape, the trapezoidal shape, the stepped-rectangular shape and a composite shape made of rectangular and trapezoidal sections. The parameter measured during the tests was the underseepage discharge since the head of the reservoir remained constant throughout. As has been mentioned the permeability coefficients of the soil materials were first determined the results of which are shown below.

Sample calculations are presented to show how the various quantities entered into the various tables contained herein were obtained.

5.1 Permeability coefficient of the foundation material (k_f) using constant-head permeameter.

Given below are the readings obtained in one run of the experiments to determine k_f.

constant head H = 90.0cm

diameter of sample D = 7.5cm

TABLE 5.1 MEASURED QUANTITIES TO DETERMINE k_f

No	Time, t(s)	Volumetric discharge (cm^3)	discharge per unit time q_o (cm^3/S)	Average q_o (cm^3/S)
1	9.4	165	17.55	
2	9.8	170	17.35	17.50
3	10.8	190	17.59	

The permeability coefficient, k_f was calculated from the expression given in Eq.(4.4) as

$$k_f = \frac{q_o L}{AH}$$

$$= \frac{4 \times 17.50 \times 18}{\pi(7.5)^2 \times 90}$$

$$= 0.079 \text{ cm/S}$$

TABLE 5.2 MEASURED QUANTITIES TO DETERMINE k_f

No	Length l(cm)	Head (cm)	Average discharge $q_o(cm^3/S)$	k_f (cm/S)
1	17.5	91.0	15.85	0.069
2	17.8	90.0	15.86	0.071
3	18.0	89.0	16.38	0.075
4	18.0	90.0	17.50	0.079
5	17.5	88.0	14.88	0.067
6	18.3	89.5	17.07	0.079
7	18.0	88.0	19.65	0.091
8	18.2	91.0	15.68	0.071
9	17.6	88.5	15.77	0.071
10	18.2	90.5	16.48	0.075

Average k_f = 0.075cm/S. This is believed to be reasonably accurate since the samples tested would be close to being representative of the entire foundation material.

5.2 Permeability coefficient of the blanket material (k_b) using the falling-head permeameter.

The following readings were obtained in the determination of k_b.

TABLE 5.3 MEASURED QUANTITIES TO DETERMINE k_b

No	H_1(cm)	H_2(cm)	Time t(s)	k_b(cm/S)
1	80	45	235	9.38×10^{-5}
2	80	40	260	1.02×10^{-4}
3	75	35	306	9.13×10^{-5}
4	70	20	485	9.91×10^{-5}
5	70	30	325	1.0×10^{-4}
6	60	40	175	8.89×10^{-5}
7	55	30	243	9.57×10^{-5}
8	50	35	155	8.81×10^{-5}
9	50	30	205	9.56×10^{-5}
10	40	25	180	1.0×10^{-4}

The coefficient of permeability k_b was evaluated from Eq.(4.11)

$$k_b = \frac{a}{A} \cdot \frac{l}{t} \cdot 2.3 \log_{10}\left(\frac{H_1}{H_2}\right)$$

Using the readings of the first run,

$$k_b = \frac{(\pi/4)(0.8)^2}{(\pi/4)(10)^2} \times \frac{6}{235} \times 2.3 \log_{10}\left(\frac{80}{45}\right)$$

$= 6.4 \times 10^{-3} \times 2.55 \times 10^{-2} \times 0.5747$

$= 9.38 \times 10^{-5}$cm/S

Similar calculations were made with the other values of H_1, H_2 and to to get 10 values of k_b entered into the table above. The average value of $k_b = 9.55 \times 10^{-4}$cm/S.

5.3 Underseepage discharge without blanket

The readings of the underseepage discharge through the foundation in the absence of any blanket are given below.

TABLE 5.4 SEEPAGE DISCHARGE IN THE ABSENCE OF A BLANKET.

No	Time t(s)	Volumetric discharge (cm^3)	Discharge per unit time (cm^3/S)
1	7.4	925	125.00
2	6.4	820	128.12
3	6.0	770	128.33
4	7.2	903	125.00
5	7.6	955	125.66
6	7.0	900	128.57
7	6.6	842	127.58
8	6.8	855	125.74
9	6.6	830	125.76
10	7.6	950	125.00

The average discharge per unit time

$$= 126.52 \text{ cm}^3/\text{s}.$$

Therefore the average discharge per unit width under the dam, Q_o

$$= \frac{126.52}{40}$$

$$= 3.16\text{cm}^3/\text{s/cm}$$

From this practical value of Q_o, the effective permeability was established as follows.

Denoting the permeability coefficient of the sand as k_f and using equation (3.9),

$$k_f = \frac{Q_o \, L_d}{hT} \qquad 5.1$$

Substituting $Q_o = 3.16 \text{ cm}^3/\text{s/cm}$, $L_d = 40\text{cm}$ h = 37cm and T = 38cm gives

$$k_f = \frac{3.16 \times 40}{37 \times 38}$$

$$= 0.09\text{cm/s}$$

Hence the modified k_f used in the subsequent calculations = 0.09cm/s for the following reasons. Obviously the absolute value of the permeability coefficient depends on the properties of water and on the characteristics of the soil. The viscosity and temperature had changed before the running of the experiments. These changes would introduce variation in the permeability of the sand. Moreover the undoubtful change in composition, magnitude and form of its grains contributed to the disparity between the theoretical and measured values of the permeability coefficients.

5.4 Underseepage discharge with the rectangular shaped blanket

TAPLE 5.5 SEEPAGE DISCHARGE UNDER THE RECTANGULAR SHAPED BLANKET.

No	Time t(s)	Volumetic discharge (cm^3)	Discharge per unit time (cm^3/S)
1	10.2	655	64.22
2	11.2	710	63.39
3	11.6	747	64.40
4	8.6	550	63.95
5	7.2	453	62.92
6	11.8	770	65.25
7	13.0	820	63.08
8	14.4	920	63.89
9	9.4	600	63.83
10	14.6	935	64.04

The average discharge per unit time

$$= 63.90 \text{ cm}^3/S$$

Hence, the average discharge per unit width under the dam, Q_m

$$= \frac{63.90}{40}$$

$$= 1.60 \text{cm}^3/S/\text{cm}$$

With reference to Figure 4.4, the following data were used to estimate the theoretical underseepage discharge under the rectangular blanket.

The length of the blanket, L_1 = 50.0cm and its uniform thickness, t = 10 cm. As in the case of the sand, the permeability coefficient, k_b of the clay was also modified for similar reasons as adduced above. The modified k_b used throughout the experiments is 1.08×10^{-4}cm/S and not 9.55×10^{-5} cm/S as was determined in the permeability test.

Using Eq.(3.11) the maximum underseepage per unit width of dam is estimated from

$$Q_m = k_f bhT \{bL_d + \tanh(bL_1)\}^{-1} \qquad 5.2$$

where

$$b = [\frac{k_b}{k_f Tt}]^{\frac{1}{2}} \qquad 5.3$$

$$= [\frac{1.08 \times 10^{-4}}{0.09 \times 38 \times 10}]^{-1}$$

$$= 1.777 \times 10^{-3} \text{ m}^{-1}$$

Substituting for all the quantities in Eq.(5.2) we get

$$Q_m = (0.09)(1.777 \times 10^{-3})(37 \times 38)\{40 \times 1.777 \times 10^{-3} + \tanh(1.77 \times 10^{-3} \times 50)\}^{-1}$$

$$= 0.2248615\{0.1596969\}^{-1}$$

$$= 1.41 \text{cm}^3/\text{S/cm}$$

This value can also be obtained from (3.13) which reads

$$Q_m/Q_o = bL_d\{bL_d + \tanh(bL_1)\}^{-1} \qquad 5.3$$

Substituting we have,

$$\frac{Q_m}{Q_o} = 1.777x10^{-3}x40\ \{1.777x10^{-3}x40+1 - \frac{2}{e^{2x1,77x10^{3}x50}+1}\}^{-1}$$

which on simplification gives

$$\frac{Q_m}{Q_o} = 0.4450931$$

or

$$Q_m = 3.16 \times 0.4450931$$

$$= 1.41cm^3/s/cm$$

The equality of the two results above confirms the relative accuracy in Q_o.

5.5 Underseepage discharge using the triangular shaped blanket

Throughout the experiments, the values of L_d, L_i, T and h remained constant irrespective of the shape of the blanket. However, the slope of the section S = 0.2 (Fig.4.5).

TABLE 5.6 SEEPAGE DISCHARGE UNDER THE TRIANGULAR SHAPED BLANKET.

No	Time t(s)	Volumetric discharge (cm^3)	Discharge per unit time (cm^3/s)
1	10.2	590	57.84
2	14.0	811	57.93
3	8.4	485	57.74
4	13.0	752	57.85
5	8.6	505	58.72
6	11.0	642	58.36

No	Time t(s)	Volumetric discharge (cm^3)	Discharge per unit time (cm^3/s)
7	10.6	610	57.55
8	14.8	860	58.11
9	15.4	895	58.12
10	5.4	320	59.26

The average discharge per unit time

$$= 58.15 cm^3/s$$

Therefore, the average discharge per unit width under the dam, Q_m

$$= \frac{58.15}{40}$$

$$= 1.45 cm^3/s/cm$$

With reference to Figure 3.2, the following practical dimensions were applicable: L_1=50.0cm, L_d=40.0cm, T=38.0cm, h=37.0cm and S=0.2. Using Eq.(3.37) the maximum underseepage discharge per unit width of dam is given by

$$\frac{Q_m}{Q_o} = L_d\left[L_d + \frac{\sqrt{L_1}\, J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}} J_{o(\tau)}}\right]^{\frac{1}{2}} \qquad 5.4$$

where

$$\alpha = \frac{k_b}{k_f} T \qquad 5.5$$

and $J_{1(\tau)}$ and $J_{o(\tau)}$ are Bessel's functions of the first kind and of the first and zeroth order respectively.

$$\tau = 2\left(\frac{\alpha L_1}{S}\right)^{\frac{1}{2}} \qquad 5.6$$

Substituting for k_b, k_f and T, in Eq.(5.5) gives

$$\alpha = \frac{1.08 \times 10^{-4}}{0.09 \times 38} = 3.15789 \times 10^{-5}$$

therefore,

$$(\alpha/S)^{\frac{1}{2}} = (\frac{3.15789 \times 10^{-5}}{0.2})^{\frac{1}{2}} = 0.012565$$

and

$$\tau = 2 \{\frac{3.15789 \times 10^{-5} \times 50}{0.2}\}^{\frac{1}{2}}$$

$$= 0.1777$$

The expansion of Bessel's function for $J_{o(\tau)}$ is given by

$$J_{o(\tau)} = 1 - \frac{\tau^2}{2^2(1!)^2} + \frac{\tau^4}{2^4(2!)^2} - \frac{\tau^6}{2^6(3!)^2} + \ldots \qquad 5.7$$

and that of $J_{1(\tau)}$ is

$$J_{1(\tau)} = \frac{\tau}{2} - \frac{\tau^3}{2^3(1!2!} + \frac{\tau^5}{2^5 2!3!} - \frac{\tau^7}{2^7 3!4!} + \ldots \qquad 5.8$$

Substituting $\tau = 0.1777$ in Eq.(5.7) gives

$$J_{o(o.1777)} = 1 - \frac{0.1777^2}{2^2 x1} + \frac{0.1777^4}{16x4} - \frac{0.1777^6}{64x36} + \ldots$$

$$= 1 - 7.89432 \times 10^{-3} + 1.558 \times 10^{-5} - 8.1996 \times 10^{-8}$$

$$= 0.9921$$

Also Eq.(5.8) gives

$$J_{1(0.1777)} = \frac{0.1777}{2} - \frac{0.1777^3}{8x1x2} + \frac{0.1777^5}{32x2x6} - \frac{0.1777^7}{128x6x24}$$

$$= 0.08885 - 3.50705 \times 10^{-4} + 4.6143 \times 10^{-7} - 3.0355 \times 10^{-10}$$

$$= 0.0885$$

Substituting these values in equation (5.4)

$$\frac{Q_m}{Q_o} = 40[40 + \frac{\sqrt{50}(0.0885)}{(0.0125656)(0.9921)}]^{-1}$$

which simplifies to

$$\frac{Q_m}{Q_o} = 40[40 + 50.198367]^{-1}$$

and on further simplification gives

$$\frac{Q_m}{Q_o} = 0.443467$$

or

$$Q_m = 3.16 \times 0.443467$$

$$= 1.40 \text{ cm}^3/\text{S/cm}$$

5.6 Underseepage discharge using the trapezoidal shaped blanket

For clarity of applicability of various dimensions the diagram of the trapezoidal blanket is given below.

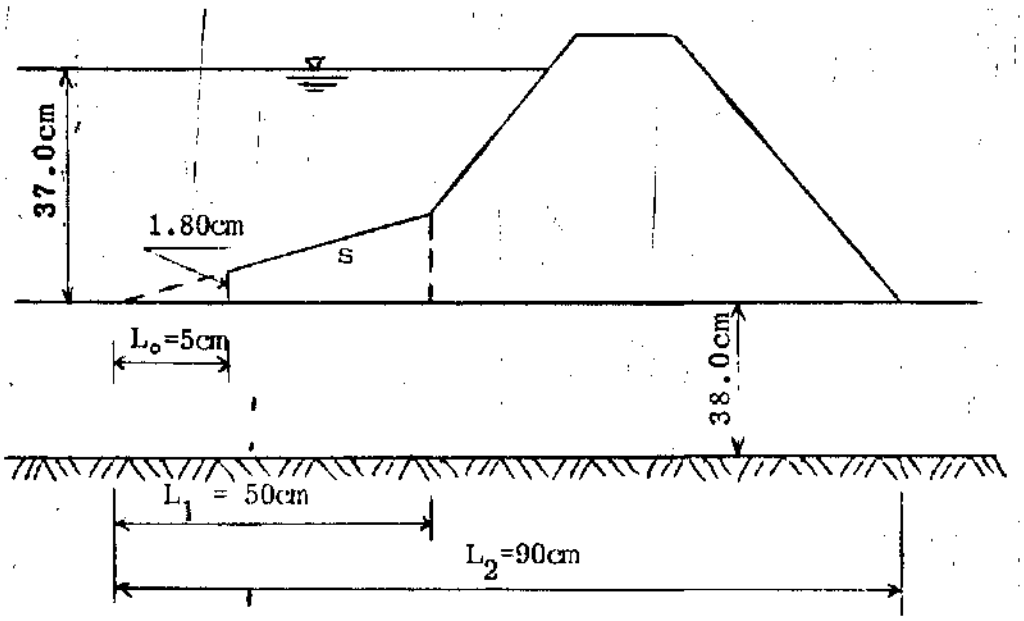


Fig.5.1 Schematic of trapezoidal blanket with applicable dimensions.

As in the previous section,

$$\alpha = 3.15789 \times 10^{-5},\ S = 0.2,\ \tau = 0.1777$$

and

$$\tau_o = 2\left(\frac{\alpha L_o}{S}\right)^{\frac{1}{2}}$$

$$= 2\left[\frac{3.15789 \times 10^{-5} \times 5}{0.2}\right]^{\frac{1}{2}}$$

$$= 0.056195$$

$$\left(\frac{\alpha}{S}\right)^{\frac{1}{2}} = 0.0125656$$

The equation for calculating the maximum underseepage under the dam is given by equation (3.62) as

$$\frac{Q_m}{Q_o} = L_d\left[L_d + \frac{\sqrt{L_1}\{J_{1(\tau)}Y_{1(\tau_o)} - Y_{1(\tau)}J_{1(\tau_o)}\}}{(\alpha/S)^{\frac{1}{2}}\{J_{o(\tau)}Y_{1(\tau_o)} + J_{1(\tau_o)}Y_{o(\tau)}\}}\right]^{-1} \qquad 5.9$$

where

$Y_{o(\tau)}$ and $Y_{1(\tau)}$ are Bessel's function of the second kind and of the zeroth and first order respectively.

The expansion of Bessel's function $Y_{o(\tau)}$ is given by

$$Y_{o(\tau)} = \frac{2}{\tau}\left[J_{o(\tau)}\left(I_n \frac{\tau}{2} + \gamma\right) + \sum_{m-1}^{\infty} \frac{(-1)^{m-1} h_m}{2^{2m}(m!)^2}(\tau)^{2m}\right] \qquad 5.10$$

where $\gamma = 0.57721566 \ldots$ is the so-called Euler constant.

and $h_m = 1 + \frac{1}{2} + \frac{1}{3} + \ldots + \frac{1}{m}$

The expansion for $Y_{1(\tau)}$ is given by

$$Y_{1(\tau)} = \frac{1}{\tau}\sum_{m=o}^{\infty} \frac{(-1)^{m-1}(h_m + h_{m+n})}{2^{2m+n}\ m!\ (m+n)!}\tau^{2m} - \frac{\tau}{\tau}^{-nn-1}\sum_{m=o} \frac{(n-m-1)!}{2^{2m-n} m!}\tau^{2m} \qquad 5.11$$

As in the previous case,

$J_{1(\tau)} = 0.0885$ and $J_{o(\tau)} = 0.9921$. Substituting $\tau_o = 0.056195$ in equation (5.8) gives

$$J_{1(\tau_o)} = \frac{0.056195}{2} - \frac{0.056195^3}{8x1x2} + \frac{0.056195^5}{32x2x6} - \ldots$$

$$= 0.0280975 - 1.1091 \times 10^{5} + 1.45934 \times 10^{-9}$$

$$= 0.028086$$

Using Eq.(5.10) and substituting $\tau = 0.1777$ gives

$$Y_{o(\tau)} = \frac{2}{\tau}[0.9921(-2.4208057 + 0.57721566)$$

$$+ \frac{(-1)^{o}(1)}{2^{2}(1!)^{2}}(0.1777)^{2} + \frac{(-1)^{1}(1.5)}{2^{4}(2!)^{2}}(0.1777)^{4}$$

$$+ \frac{(-1)^{2}(1.8333)}{2^{6}(3!)^{2}}(0.1777)^{6}]$$

which upon simplification gives

$$Y_{o(\tau)} = \frac{2}{\tau}[(-1.8290257) + 0.00789432 - 2.33701\times10^{-5}$$

$$+ 2.50543 \times 10^{-8}]$$

and further simplification results in

$$Y_{o(\tau)} = \frac{2}{\tau}[-1.821154]$$

$$= -1.15938$$

Substituting $\tau = 0.1777$ in equation (5.11) gives

$$Y_{1}(0.1777) = \{\frac{(-1)^{-1}(0+1)}{2^{1}0!(1!)}(0.1777)^{o} + \frac{(-1)^{o}(1+1.5)}{2^{3}1!2!}(0.1777)^{2}$$

$$+ \frac{(-1)^1(1.5+1.8333)}{2^5\ 2!\ 3!}(0.1777)^4 + \frac{(-1)^2(1.8333 + 2.08333)}{2^7\ 3!\ 4!}(0.1777)^6$$

$$+ \frac{(-1)^3(2.08333 + 2.28333)}{2^9\ 4!\ 5!}(0.1777)^8 + \ldots\}$$

$$- \frac{0.1777^{-1}}{\tau}\{\frac{(1-0-1)!}{2^{-1}0!}(0.1777)^0\}$$

Simplifying, we get

$$Y_{1(0.1777)} = \frac{0.1777}{\tau}\{-\frac{1}{2} + 4.93395 \times 10^{-3} - 8.6556 \times 10^{-6}$$

$$+ 6.69065 \times 10^{-9} - 2.94432 \times 10^{-12}\}$$

$$- \frac{1}{0.1777\tau}\{2\}$$

$$= \frac{0.1777}{\tau}\{- 0.4950746\} - \{\frac{2}{0.1777\tau}\}$$

which finally simplifies to

$$Y_{1(\tau)} = - 0.0280032 - 3.5825535$$

$$= - 3.6105568$$

Also, $Y_{1(\tau_o)} = Y_{1(0.05195)}$ gives, using Eq.(5.11)

$$\frac{0.05195}{\pi}\{\frac{(-1)^{-1}}{2!0!(1!)}(0.05195)^0 + \frac{(-1)^0(1+1.5)}{2^3\ 1!\ 2!}(0.05195)^2$$

$$+ \frac{(-1)^1(1.5+1.8333)}{2^5\ 2!\ 3!}(0.05195)^4 + (-1)^2(1.8333+2.08333)(0.05195)^6\}$$

$$- \frac{0.05195^{-1}}{\pi}\{\frac{(1-0-1)!}{2^{-1}\ 0!}(0.05195)^0\}$$

Simplify to get

$$Y_{1(\tau_o)} = \frac{0.05195}{\tau}\{-\frac{1}{2} + 4.21687 \times 10^{-4} - 6.32251 \times 10^{-8} + 4.17693 \times 10^{-12}\} - \frac{2}{0.05195\tau}$$

$$= \frac{0.05195}{\tau}\{-0.4995783\} - \{\frac{2}{0.05195\tau}\}$$

which finally gives

$$Y_{1(\tau_o)} = -12.2627$$

Substituting for $J_{1(\tau_o)}$, $Y_{1(\tau_o)}$, $J_{o(\tau)}$, $Y_{1(\tau)}$, $J_{1(\tau)}$ and $J_{o(\tau)}$ in Eq.(5.9) gives

$$\frac{Q_m}{Q_o} = 40[40 + \frac{\sqrt{50}\{0.0885(-12.2627)-(-3.6106)(0.0281)\}}{0.0126\{0.9921(-12.2627)+(0.0281)(-1.1594)\}}]^{-1}$$

which simplifies to

$$\frac{Q_m}{Q_o} = 40[40 + \frac{\sqrt{50}(-0.9837)}{0.0126(-12.1984)}]^{-1}$$

$$= 40[40 + 45.2557]^{-1}$$

and finally we get

$$\frac{Q_m}{Q_o} = 0.4692$$

So that

$$Q_m = 3.16 \times 0.4692$$

$$= 1.48 cm^3/s/cm$$

TABLE 5.7 SEEPAGE DISCHARGE UNDER THE TRAPEZOIDAL SHAPED BLANKET.

No	Time t(s)	Volumetric discharge (cm^3)	Discharge per unit time (cm^3/s)
1	10.2	629	61.66
2	10.0	622	62.20
3	14.0	877	62.64
4	11.6	720	62.07
5	15.2	943	62.04
6	13.0	809	62.23
7	9.0	562	62.44
8	11.8	736	62.37
9	11.4	706	61.93
10	15.4	957	62.14

The average discharge per unit time = $62.17 cm^3/s$.

Hence, the mean practical seepage discharge per unit width of dam, Q_m

$$= \frac{62.17}{40}$$

$$= 1.55 cm^3/s/cm$$

5.7 Underseepage discharge using a stepped-rectangular blanket

TABLE 5.8 SEEPAGE DISCHARGE UNDER THE STEPPED-RECTANGULAR SHAPED BLANKET.

No	Time t(s)	Volumetric discharge (cm^3)	Discharge per unit time (cm^3/s)
1	10.2	700	68.63
2	11.8	800	67.80
3	12.6	853	67.70
4	10.2	695	68.14
5	13.0	877	67.46
6	9.6	655	68.23
7	13.6	920	67.65
8	7.8	533	68.33
9	9.0	613	68.11
10	11.4	778	68.25

The average discharge per unit time = $68.03 cm^3/s$.

Therefore, the average maximum underseepage discharge under the blanket, Q_m

$$= \frac{68.03}{40}$$

$$= 1.70 cm^3/s/cm$$

The theoretical seepage discharge using the stepped-rectangular blanket is now determined. For convenience of analysis, the diagram of the blanket as experimented with is given below.

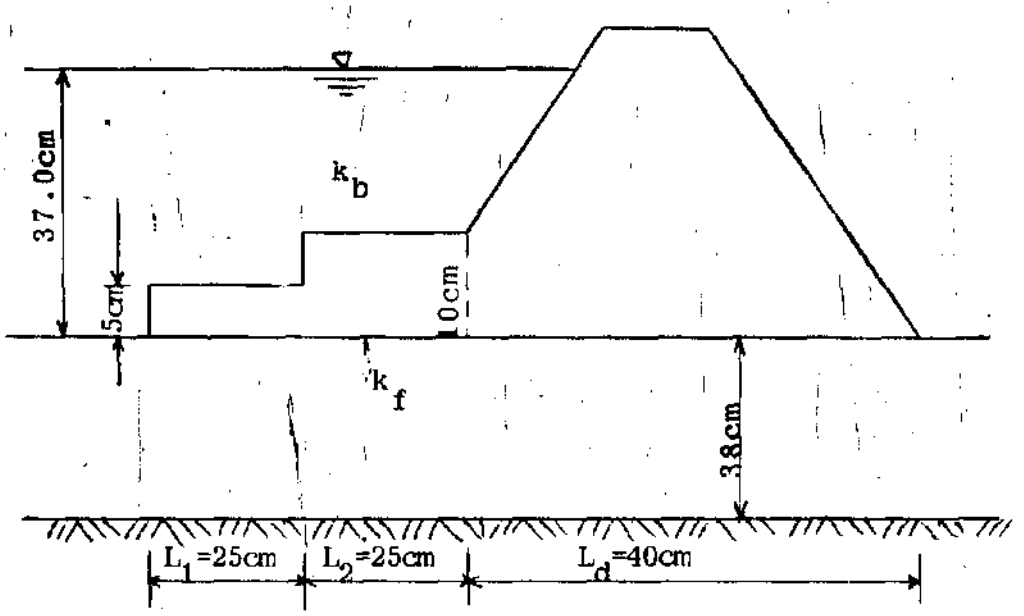


Fig.5.2 Schematic of stepped-rectangular blanket with the numerical dimensions.

With reference to Figure 5.2, we see that $L_1=L_2=25.0$cm, $t_1=5$cm and $t_2=10$cm. Given that

$$b_1=\left(\frac{k_b}{k_f T t_1}\right)^{\frac{1}{2}} = \left(\frac{1.08 \times 10^{-4}}{0.09 \times 38 \times 5}\right)^{\frac{1}{2}} = 2.513 \times 10^{-3}$$

$$b_2 = \left(\frac{k_b}{k_f T t_2}\right)^{\frac{1}{2}} = \left(\frac{1.08 \times 10^{-4}}{0.09 \times 38 \times 10}\right)^{\frac{1}{2}} = 1.777 \times 10^{-3}$$

and

$$b_1 + b_2 = 4.29 \times 10^{-3}$$

the maximum seepage discharge is given by

$$Q_m=k_f(b_1+b_2)hT\{(b_1+b_2)L_d + \tanh(bL_1)+\tanh(bL_2)\}^{-1} \qquad 5.12$$

Substituting for all the quantities in equation(5.12)

$$Q_m=0.09(4.29\times10^{-3})(37\times38)\{40\times4.29\times10^{-3} +$$

$$\tanh(2.5131\times10^{-3}\times 25) + \tanh(1.777 \times 10^{-3}\times25)\}^{-1}$$

Simplifying, we obtain,

$$Q_m = 0.5429\{0.1716 + \tanh(0.0628)+\tanh(0.0444)\}^{-1}$$

or

$$Q_m = 0.5429\{0.1716 + (1- \frac{2}{e^{2x0.0628}+1})+(1- \frac{2}{e^{2x0.0444}+1})\}^{-1}$$

which is evaluated to

$$Q_m = 0.5429\ \{0.1716 + 0.0627 + 0.0444\}^{-1}$$

$$= 0.5429(02787)^{-1}$$

$$= 1.95 cm^3/s/cm$$

This result may also be obtained from the equation

$$\frac{Q_m}{Q_o} = (b_1+b_2)L_d[(b_1+b_2)L_d + \tanh(b_1L_1) + \tanh(b_2L_2)]^{-1} \qquad 5.13$$

$$= 4.29 \times 40\ [4.29 \times 40 + 0.0627 + 0.0444]^{-1}$$

$$= 0.1716[0.2787]^{-1} = 0.6157$$

so that

$$Q_m = 3.16 \times 0.6157$$

$$= 1.94\ cm^3/s/cm$$ again showing relative accuracy in the Q_o value.

5.8 Underseepage discharge using the rectangular-trapezoidal blanket

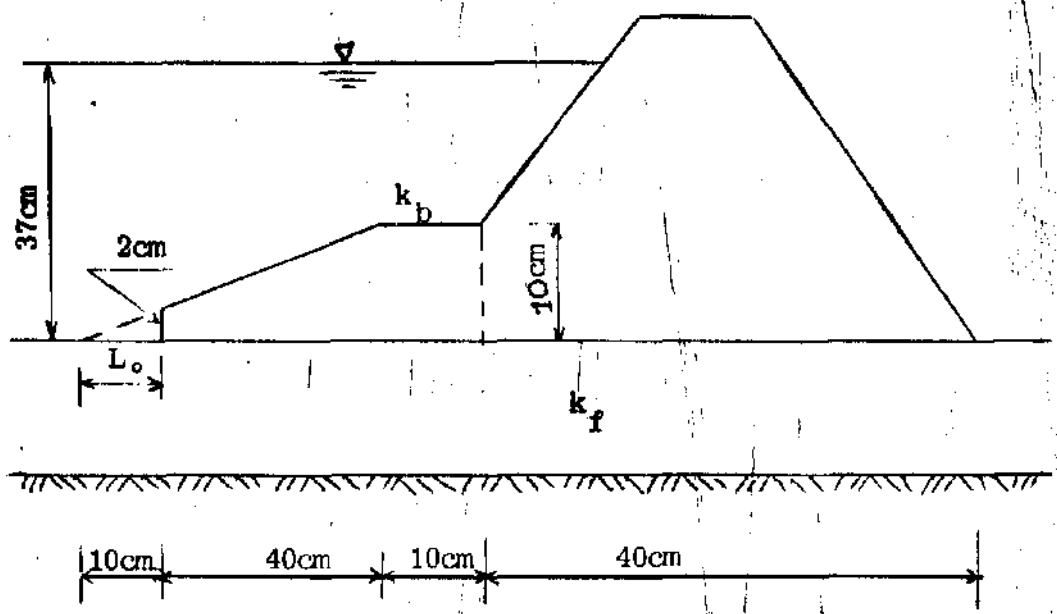


Fig.5.3 Schematic of rectangular-trapezoidal blanket with the practical dimensions.

With reference to Figure 5.3 we have that L_o=10cm, S=0.2 and L_1=60cm.

We recall from section 5.5 that

$$\alpha = 3.15789 \times 10^{-5} \text{ and}$$

$$(\frac{\alpha}{S})^{\frac{1}{2}} = 0.01256$$

Using the above dimensions, we have

$$\tau = 2(\frac{\alpha L_1}{S})^{\frac{1}{2}} = 2(\frac{3.15789 \times 10^{-5} \times 60}{0.2}) = 0.1947$$

and

$$\tau_o = 2(\frac{\alpha L_o}{S})^{\frac{1}{2}} = 2(\frac{3.15789 \times 10^{-5} \times 10}{0.2})$$

$$= 0.0795$$

Using equation (5.8)

$$J_{1(\tau)} = J_{1(0.1947)}$$

$$= \frac{0.1947}{2} - \frac{0.1947^3}{2^3\,1!2!} + \frac{0.1947^5}{2^5 2!\,3!} - \ldots$$

$$= 0.09735 - 4.61294 \times 10^{-4} + 7.286 \times 10^{-7}$$

$$= 0.0969$$

Also,

$$J_{1(\tau_o)} = J_{1(0.0795)}$$

$$= \frac{0.0795}{2} - \frac{0.0795^3}{2^3 1!2!} + \frac{0.0795^5}{2^5\,2!3!} - \ldots$$

$$= 0.03975 - 3.14 \times 10^{-5} + 8.27 \times 10^{-9}$$

$$= 0.0397$$

Using Eq.(5.7)

$$J_{o(\tau)} = 1.0 - \frac{0.1947^2}{2^2} + \frac{0.1947^4}{2^4(4)} - \frac{0.1947^6}{2^6(3!)^2}$$

$$= 1 - 9.477 \times 10^{-3} + 2.245 \times 10^{-5} - 2.364 \times 10^{-8}$$

$$= 0.9905$$

Substituting $\tau = 0.1947$ in Eq.(5.10) gives

$$Y_{o(\tau)} = \frac{2}{\pi}\{0.9905(-2.3294425 + 0.57721566) + \frac{(-1)^o(1)}{2^2(1!)^2}(0.1947)^2$$

$$+ \frac{(-1)^1(1.5)}{2^4(2!)^2}(0.1947)^4 + \frac{(-1)^2(1.833)}{2^6(3!)^2}(0.1947)^6\}$$

which on simplification gives

$$Y_{o(\tau)} = \frac{2}{\pi}\{-1.7355807 + 9.47702 \times 10^{-3} - 3.36802 \times 10^{-5}$$

$$+ 4.33465 \times 10^{-8}\}$$

and further simplification results in

$$Y_{o(\tau)} = \frac{2}{\pi}\{-1.7261373\}$$

$$\doteq -1.0989$$

Using Eq.(5.11) and substituting τ=0.1947 gives

$$Y_{1(0.1947)} = Y_{1(\tau)}$$

$$= \frac{0.1947}{\tau}\left\{\frac{(-1)^{-1}(1)}{2^1\ 0!(1!)}(0.1947)^0 + \frac{(-1)^0(1+1.5)}{2^3\ 1!2!}(0.1947)^2\right.$$

$$\left.\frac{(-1)^1(1.5+1.8333)}{2^5\ 2!\ 3!}(0.1947)^4 + \frac{(-1)^2(1.8333+2.0833)}{2^7\ 3!\ 4!}(0.1947)^6\right\}$$

$$-\left(\frac{2}{0.1947\tau}\right)$$

which upon simplification gives

$$Y_{1(\tau)} = \frac{0.1947}{\tau}\{-\frac{1}{2} + 5.92313 \times 10^{-3} - 1.24741 \times 10^{-5}$$

$$+ 1.15755 \times 10^{-8}\} - 3.2697471$$

$$\doteq -3.30037$$

Similarly $Y_{1(\tau_o)} = Y_{1(0.0795)}$ using Eq.(5.11) gives

$$Y_{1(\tau_o)} = \frac{0.0795}{\tau}\{-\frac{1}{2} + 0.15625(0.0795)^2$$

$$- 8.68055 \times 10^{-3}(0.0795)^4\} - \left\{\frac{2}{0.0795\tau}\right\}$$

which simplifies to

$$Y_{1(\tau_o)} = \frac{0.0795}{\tau}\{-\frac{1}{2} + 9.87539 \times 10^{-4} - 3.46749 \times 10^{-7}\}$$

$$- 8.0077958$$

which finally yields

$$Y_{1(\tau_o)} = -\ 8.02042$$

Using Eq.(3.87) the maximum underseepage discharge can be calculated from

$$\frac{Q_m}{Q_o} = L_d\left[L_d + \frac{\sqrt{L_1}\{J_{1(\tau)}Y_{1(\tau_o)} - Y_{1(\tau)}J_{1(\tau_o)}\}}{(\alpha/S)^{\frac{1}{2}}\{J_{o(\tau)}Y_{1(\tau_o)}+J_{1(\tau_o)}Y_{o(\tau)}\}}\right]^{-1}$$

$$+ bL_d[bL_d + \tanh(bL_1)]^{-1} \qquad 5.14$$

Substituting for all the quantities in Eq.(5.14) gives

$$\frac{Q_m}{Q_o} = 40\left[40 + \frac{\sqrt{60}\{0.0969(-8.0204) - (-3.30037)(0.0397)\}}{0.01256\{0.9905(-8.0204)+(0.0397)(-1.0989\}}\right]^{-1}$$

$$+ 1.777 \times 10^{-3} \times 40\{1.777 \times 10^{-3} \times 40$$

$$+ \left(1 - \frac{2}{e^{2\times1.777\times10^{-3}\times60}+1}\right)\}^{-1}$$

Upon simplification, we have

$$\frac{Q_m}{Q_m} = 40\left[40 + \frac{\sqrt{60}\{-0.6461\}}{0.01256\{-7.9878\}}\right]^{-1}$$

$$+ 0.07108\ \{0.07108 + 0.1062\}^{-1}$$

and further simplification results in

$$\frac{Q_m}{Q_o} = 40[40 + 77]^{-1} + 0.07108\ \{0.1772\}^{-1}$$

and finally we get

$$\frac{Q_m}{Q_o} = 0.342 + 0.401$$

$$= 0.743$$

so that

$$Q_m = 3.16 \times 0.743$$

$$= 2.35 cm^3/s/cm$$

TABLE 5.9 SEEPAGE DISCHARGE UNDER THE RECTANGULAR-TRAPEZOIDAL SHAPED BLANKET.

No	Time t(s)	Volumetric discharge (cm^3)	Discharge per unit time (cm^3/s)
1	8.0	550	68.75
2	8.6	612	71.15
3	9.4	653	69.50
4	10.0	716	71.60
5	5.6	396	70.71
6	8.6	591	68.73
7	4.4	318	72.27
8	7.2	515	71.53
9	10.0	692	69.20
10	9.0	633	70.33

The average discharge per unit time = 70.4 cm^3/s

Hence, the discharge per unit width

$$= \frac{70.4}{40}$$

$$= 1.76 cm^3/s/cm$$

5.9 Other seepage quantities

Using the known underseepage discharge values, some other seepage quantities are here determined.

(i) Uplift forces (F)

Uplift force is the upward force of water as it seeps or flows through the foundation or dam, causing a reduction in the effective weight of the structure above it. It is often assumed that the uplift pressure varies linearly from full hydrostatic pressure at the upstream face (heel) to full tailwater pressure at the downstream face (toe). Hence, generally,

$$F = \gamma \frac{(h_1+h_2)}{2} L_d \qquad 5.15$$

when there is no tailwater like in the case of this study, h_2=o and f becomes

$$F = \frac{\gamma h L_d}{2} \qquad 5.16$$

which is the uplift force per unit length under the dam in the absence of any blanket.

Substituting the experimental values into Eq.(5.16) gives

$$F = \frac{10^{-3} \times 981 \times 37 \times 40}{2}$$

$$= 726N$$

We will now consider the blanket shapes in turn and determine the uplift force in each case.

(a) Rectangular blanket

The seepage force under the dam using rectangular blanket is given by equation (3.21) as

$$F = 0.5\ \gamma bhL_d^2\{bL_d + \tanh\ (\psi L_1^{1.5}\}^{-1} \qquad 5.17$$

where $b = 1.777 \times 10^{-3}$

$$\psi = \left(\frac{k_b}{k_f T \forall}\right)^{\frac{1}{2}} \qquad 5.18$$

and $\forall = tL_1$ = volume of blanket material per unit width.

Substituting for t and L_1 we get

$$\forall = 10 \times 50 = 500 cm^3/cm$$

therefore, $\psi = \left(\frac{1.08 \times 10^{-4}}{0.09 \times 38 \times 500}\right)^{\frac{1}{2}}$

$$= 2.51 \times 10^{-4}$$

Substituting for the quantities in equation (5.17) gives

$$F = 0.5 \times 10^{-3} \times 981 \times 1.777 \times 10^{-3} \times 37 \times 40\{40 \times 1.777 \times 10^{-3} + \tanh(2.51 \times 10^{-4} \times 50^{1.5})\}^{-1}$$

Simplification of which gives

$$F = 323N$$

Eq.(5.17) can be expressed in terms of the seepage discharge as follows

$$F = \frac{Q_{in}}{Q_o}\left(\frac{\gamma hL_d}{2}\right) \qquad 5.19$$

Substituting the computed seepage discharge under the rectangular blanket in equation (5.19) yields

$$F = \frac{1.41}{3.16}\left(\frac{10^{-3} \times 981 \times 37 \times 40}{2}\right)$$

$$= 324N$$

which represents the theoretical seepage uplift force. Substituting the measured value of the seepage discharge into Eq.(5.18) gives the actual uplift force, F_{act}. Thus, we have

$$F_{act} = \frac{1.60}{3.16}\left(\frac{10^{-3} \times 981 \times 37 \times 40}{2}\right)$$

$$= 367.6N$$

(b) Triangular blanket

Using the calculated seepage discharge Q_m in Eq.(5.18) gives the theoretical uplift force as

$$F = \frac{1.40}{3.16}\left(\frac{10^{-3} \times 981 \times 37 \times 40}{2}\right)$$

$$= 321.6N$$

The actual uplift force is then given by

$$F_{act} = \frac{1.45}{3.16} \times \left(\frac{10^{-3} \times 981 \times 37 \times 40}{2}\right)$$

$$= 333N$$

(c) Trapezoidal blanket

The predicted uplift force is given by

$$F = \frac{1.48}{3.16}\left(\frac{10^{-3} \times 981 \times 37 \times 40}{2}\right)$$

$$= 340N$$

The observed uplift force is

$$F_{act} = \frac{1.55}{3.16}\left(\frac{10^{-3}\times 981\times 37\times 40}{2}\right)$$

$$= 348.8N$$

(d) Stepped-rectangular blanket

The theoretical uplift force is given by

$$F = \frac{1.95}{3.16}\left(\frac{10^{-3}\times 981\times 37\times 40}{2}\right)$$

$$= 448N$$

and the actual force by

$$F_{act} = \frac{1.70}{3.16}\left(\frac{10^{-3} \times 981 \times 37 \times 40}{2}\right)$$

$$= 390.6N$$

(3) Rectangular-trapezoidal blanket

The calculated uplift force is given by

$$F = \frac{2.35}{3.16}\left(\frac{10^{-3}\times 981\times 37\times 40}{2}\right)$$

$$= 540N$$

The actual uplift force is evaluated from

$$F_{act} = \frac{1.76}{3.16}\left(\frac{10^{-3}\times 981\times 37\times 40}{2}\right)$$

$$= 404.3N$$

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Difference in theoretical and experimental values of the forces are explained herein later.

(ii) Seepage discharge velocity

The quantity of seepage divided by the entire cross-sectional area including voids and solids and normal

to the direction of flow, gives the discharge velocity under a given hydraulic gradient. The seepage discharge velocity, V_d in the absence of any blanket is evaluated from

$$V_d = \frac{Q_o}{A} \qquad 5.20$$

$$= \frac{3.16 \times 40}{38 \times 50}$$

$$= 0.07 cm/S$$

It should be noted that the discharge velocity is an imaginary velocity that does not exist anywhere (Cedergren, 1977).

5.10 Dependence of Q_m on α and Ψ

Analysis is here given to see how the quantity of seepage discharge varies with $\alpha(= \frac{k_b}{k_f T})$ and Ψ $(=tL_1)$ for different shapes of the blanket which provides a means of graphical comparison as to the efficiency of each shape of the blanket.

That α and Ψ contain all the properties and dimensions of the soil materials presents the optimal performance of the blanket shapes. We will consider the optimal performance theoretically using the existing equations and then experimental using the results of the experiments.

(i) Theoretical optimal performance of blankets

We will eliminate b and L_1 in equation (5.3) using equation (5.18) and given that

$$L_1 = 1.263(\Psi/\alpha)^{1/3} \qquad 5.21$$

We rewrite Eq.(5.3) for easy reference

$$\frac{Q_m}{Q_o} = bL_d\{bL_d + \tanh(bL_1)\} \qquad 5.22$$

Now $b = \psi L_1^{\frac{1}{2}} = \psi\sqrt{L_1}$ and 5.23

equation (5.18) reads

$$\psi = \left(\frac{k_b}{k_f T\ \forall}\right)^{\frac{1}{2}} \qquad 5.24$$

Since $\alpha = \dfrac{k_b}{k_f T}$

then $T = \dfrac{k_b}{k_f \alpha}$

Substituting for T in equation (5.24) gives

$$\psi = \left(\frac{k_b}{k_f x \frac{k_b}{k_f}\alpha x \forall}\right)^{\frac{1}{2}}$$

from which

$$\psi = \left(\frac{\alpha}{\forall}\right)^{\frac{1}{2}}$$

Substitute for ψ and L_1 in Eq.(5.23) to get

$$b = \left(\frac{\alpha}{\forall}\right)^{\frac{1}{2}} \sqrt{1.263}\left(\frac{\forall}{\alpha}\right)^{1/3} \qquad 5.25$$

Substituting for b and L_1 in Eq.(5.22) yields

$$\frac{Q_m}{Q_o} = \left(\frac{\alpha}{\forall}\right)^{\frac{1}{2}}\left[1.263\left(\frac{\forall}{\alpha}\right)^{\frac{1}{3}}\right]^{\frac{1}{2}} L_d \left\{\left(\frac{\alpha}{\forall}\right)^{\frac{1}{3}}\left[1.263\left(\frac{\forall}{\alpha}\right)^{\frac{1}{3}}\right]^{\frac{1}{2}} L_d + \tanh\left(\left(\frac{\alpha}{\forall}\right)^{\frac{1}{2}}\left[1.263\left(\frac{\forall}{\alpha}\right)^{\frac{1}{3}}\right]^{\frac{1}{2}} x\ 1.263\left(\frac{\forall}{\alpha}\right)^{\frac{1}{3}}\right)\right\}^{-1}$$

from which

$$\frac{Q_m}{Q_o} = \sqrt{1.263}\left(\frac{\alpha}{\forall}\right)^{\frac{1}{3}} L_d \left\{\sqrt{1.263}\left(\frac{\alpha}{\forall}\right)^{\frac{1}{3}} L_d + \tanh\left(1.263^{1.5}\right)\right\}^{-1}$$

$$= \frac{\sqrt{1.263}\ \left(\frac{\alpha}{\forall}\right)^{\frac{1}{3}} L_d}{\sqrt{1.263}\left(\frac{\alpha}{\forall}\right)^{\frac{1}{3}} L_d + \tanh(1.4194)}$$

Invert to get

$$\frac{Q_o}{Q_m} = \frac{\sqrt{1.263}(\frac{\alpha}{\nu})^{\frac{1}{3}}L_d + 0.8895}{\sqrt{1.263}(\frac{\alpha}{\nu})^{\frac{1}{3}}L_d}$$

which simplifies to

$$\frac{Q_o}{Q_m} = 1 + \frac{0.8895}{\sqrt{1.263}(\frac{\alpha}{\nu})^{\frac{1}{3}}L_d}$$

and finally we obtain

$$\frac{Q_o}{Q_m} = 1 + \frac{0.7915}{L_d(\frac{\alpha}{\nu})^{\frac{1}{3}}} \qquad 5.26$$

Similar analyses were made for the remaining shapes and the results showed that

$$\frac{Q_o}{Q_m} = 1 + \frac{0.8953}{L_d\ (\frac{\alpha}{\nu})^{\frac{1}{3}}} \qquad 5.27$$

for the triangular blanket.

That of the trapezoidal blanket is

$$\frac{Q_o}{Q_m} = 1 + \frac{0.8447}{L_d\ (\frac{\alpha}{\nu})^{\frac{1}{3}}} \qquad 5.28$$

The analysis for the stepped-rectangular blanket gave

$$\frac{Q_o}{Q_m} = 1 + \frac{0.7686}{L_d\ (\frac{\alpha}{\nu})^{\frac{1}{3}}} \qquad 5.29$$

and lastly, we have

$$\frac{Q_o}{Q_m} = 1 + \frac{0.7515}{L_d \left(\frac{\alpha}{\Psi}\right)^{\frac{1}{3}}} \qquad 5.30$$

for the rectangular-trapezoidal blanket.

The above results were used to calculate the values given in Table 5.1 below.

TABLE 5.1C DATA FOR PLOTTING FIGURE 5.4.

	$\frac{Q_m}{Q_o}$				
$L_d(\frac{\alpha}{\Psi})^{\frac{1}{3}}$	Rectangular	Triangular	Trapezoidal	Stepped-rectangular	Rectangular-trapezoidal
0.0	0.0	0.0	0.0	0.0	0.0
0.2	0.202	0.183	0.191	0.206	0.210
0.4	0.336	0.308	0.321	0.342	0.347
0.6	0.431	0.401	0.415	0.438	0.444
0.8	0.503	0.472	0.486	0.510	0.516
1.0	0.558	0.528	0.542	0.565	0.571
1.2	0.603	0.573	0.587	0.609	0.615
1.4	0.639	0.610	0.624	0.646	0.651
1.6	0.669	0.641	0.654	0.676	0.680
1.8	0.695	0.668	0.681	0.701	0.705
2.0	0.716	0.691	0.700	0.722	0.727
2.2	0.735	0.711	0.722	0.741	0.745
2.4	0.752	0.728	0.740	0.757	0.762

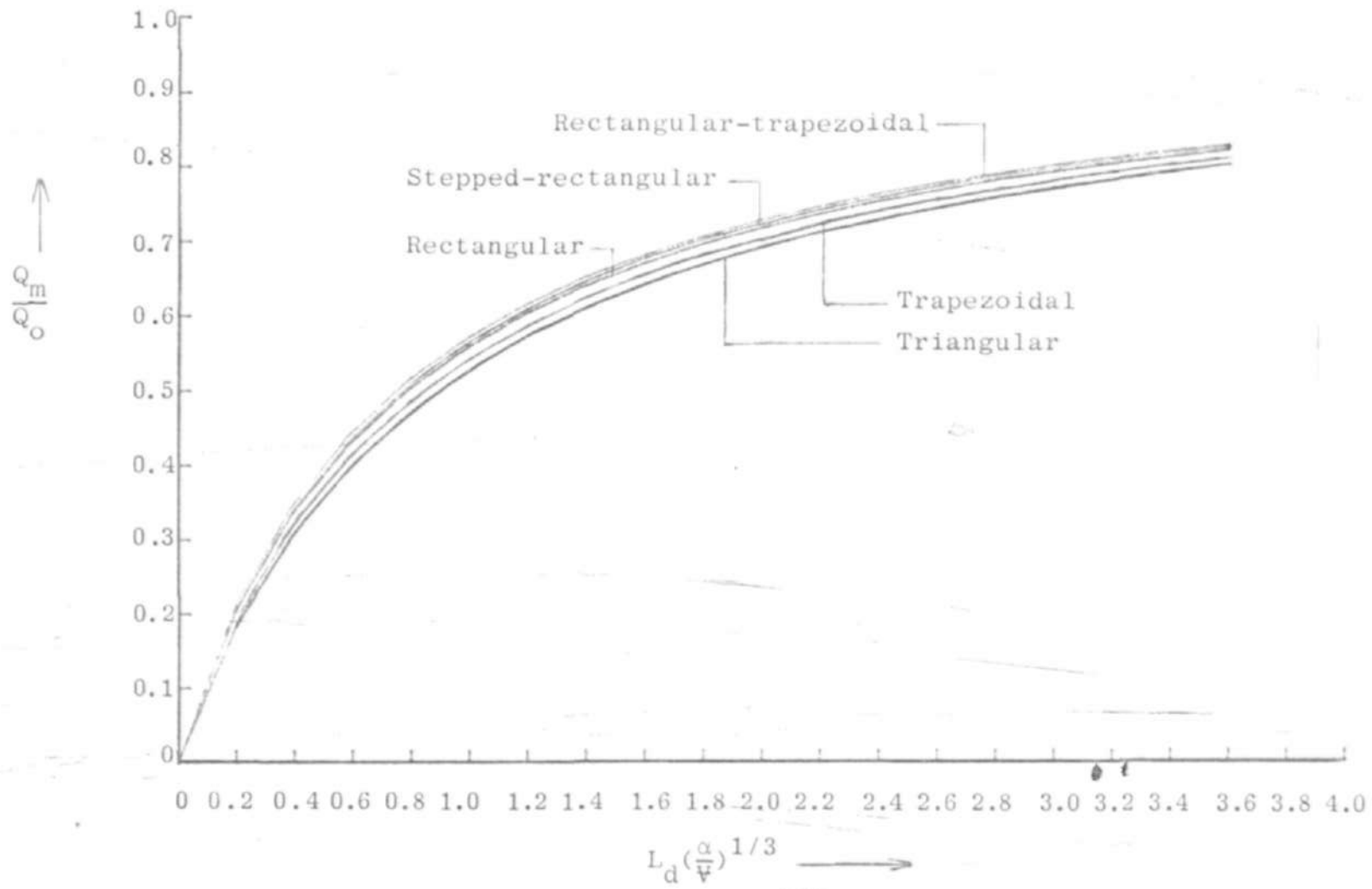


Fig.5.4 Plot of Q_m/Q_o against $L_d(\frac{\alpha}{\forall})^{1/3}$ for theoretical optimal performance of blankets.

2.6	0.766	0.744	0.755	0.772	0.776
2.8	0.780	0.758	0.768	0.785	0.788
3.0	0.791	0.770	0.780	0.796	0.800
3.2	0.802	0.781	0.791	0.806	0.810
3.4	0.811	0.792	0.801	0.816	0.819
3.6	0.820	0.801	0.810	0.824	0.827

(ii) Experimental optimal performance of the blankets

Consider the generalized equation

$$\frac{Q_o}{Q_m} = 1 + \frac{C}{L_d(\frac{\alpha}{V})^{\frac{1}{3}}} \qquad 5.31$$

The constant C in Eq.(5.31) was found to deviate from those of the theoretical values respectively for the same reasons accounting for differences between the theoretical and experimental values.

Experimental values were then substituted in the equation, for each blanket shape to determine the functional or effective constant for that shape. Results showed that

$$\frac{Q_o}{Q_m} = 1 + \frac{0.156}{L_d(\frac{\alpha}{V})^{\frac{1}{3}}} \qquad 5.32$$

for the rectangular blanket.

For the triangular blanket we have

$$\frac{Q_o}{Q_m} = 1 + \frac{0.2358}{L_d(\frac{\alpha}{V})^{\frac{1}{3}}} \qquad 5.33$$

The results of the calculation for the trapezoidal shape was found to be

$$\frac{Q_o}{Q_m} = 1 + \frac{0.2047}{L_d(\frac{\alpha}{\Psi})^{\frac{1}{3}}} \qquad 5.34$$

and that of the stepped-rectangular blanket gave

$$\frac{Q_o}{Q_m} = 1 + \frac{0.1503}{L_d(\frac{\alpha}{\Psi})^{\frac{1}{3}}} \qquad 5.35$$

Finally, the result for the rectangular-trapezoidal blanket was

$$\frac{Q_o}{Q_m} = 1 + \frac{0.1439}{L_d(\frac{\alpha}{\Psi})^{\frac{1}{3}}} \qquad 5.36$$

TABLE 5.11 DATA FOR PLOTTING FIGURE 5.5.

	$\frac{Q_m}{Q_o}$				
$L_d(\frac{\alpha}{\Psi})^{\frac{1}{3}}$	Rectangular	Triangular	Trapezoidal	Stepped-rectangular	Rectangular trapezoidal
0.0	0.0	0.0	0.0	0.0	0.0
0.2	0.562	0.459	0.494	0.571	0.582
0.4	0.719	0.629	0.661	0.727	0.735
0.6	0.794	0.718	0.746	0.800	0.807
0.8	0.837	0.772	0.796	0.842	0.848
1.0	0.865	0.809	0.830	0.869	0.874
1.2	0.885	0.836	0.854	0.889	0.893
1.4	0.900	0.855	0.872	0.903	0.906
1.6	0.911	0.872	0.887	0.914	0.917
1.8	0.920	0.884	0.898	0.923	0.926
2.0	0.928	0.894	0.907	0.930	0.933

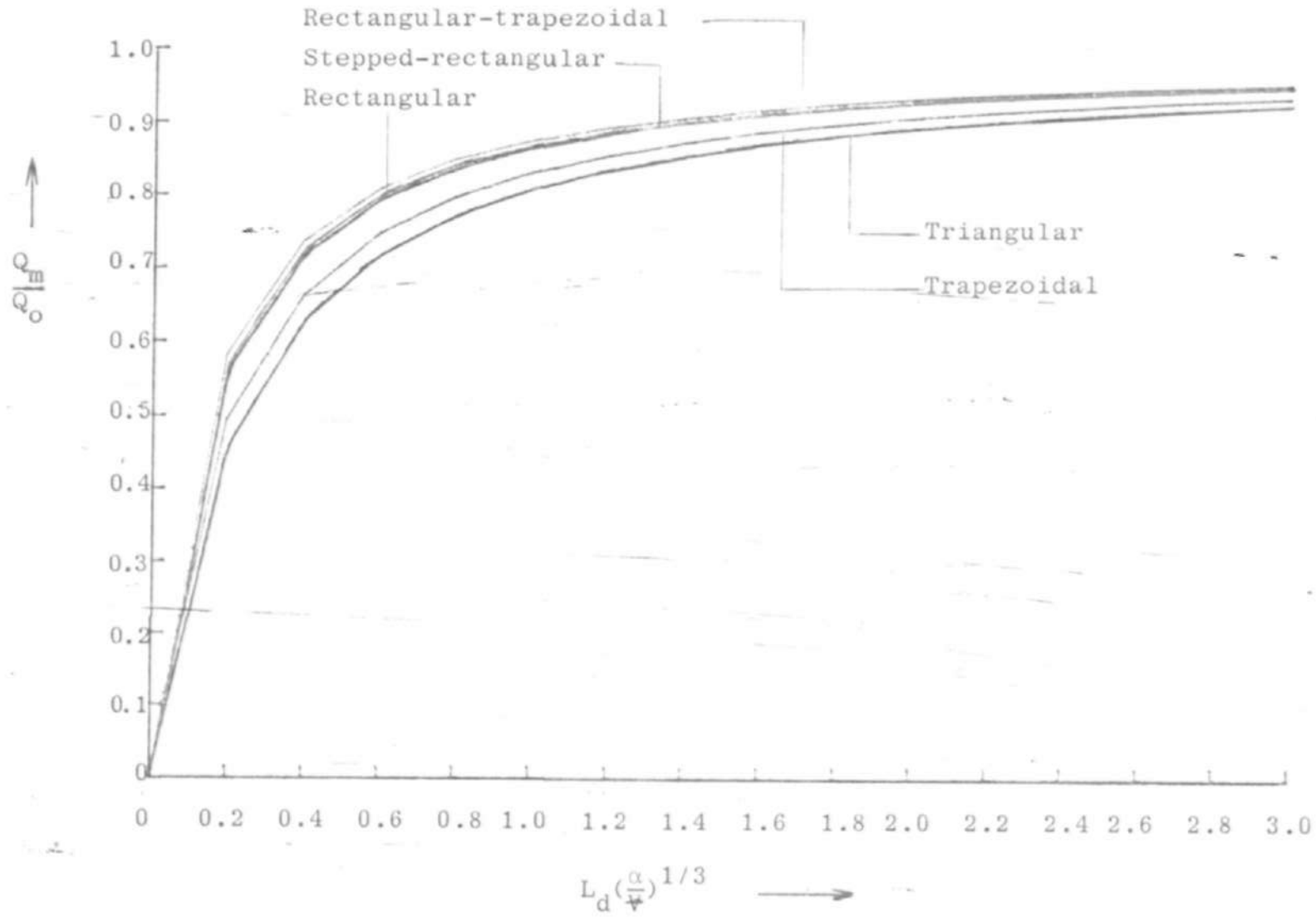


Fig.5.5 Plot of Q_m/Q_o against $L_d(\frac{\alpha}{\forall})^{1/3}$ for the experimental performance of blankets.

2.2	0.934	0.903	0.915	0.936	0.939
2.4	0.939	0.911	0.921	0.941	0.943
2.6	0.943	0.917	0.927	0.945	0.948
2.8	0.947	0.922	0.932	0.949	0.951
3.0	0.951	0.927	0.936	0.952	0.954

Table 5.11 Data for plotting Figure 5.5

5.11 Dependence of the seepage force F on the discharge Q_m

For a constant upstream head and in the absence of tailwater, a plot was made of the seepage force against the quantity of seepage discharge to study the relationship between them. The graph presented a linear relationship meaning that increase in the seepage discharge quantity would produce a corresponding increase in the uplift force.

TABLE 5.12 DISCHARGE QUANTITIES AND THE CORRESPONDING UPLIFT FORCES.

Shape of blanket	Discharge quantity ($cm^3/S/cm$)		Uplift Force (N)	
	Actual discharge	theoretical discharge	Actual force	theoretical force
Rectangular	1.60	1.41	367.6	323
Triangular	1.45	1.40	333	321
Trapezoidal	1.55	1.48	348.8	340
Stepped-rectangular	1.70	1.95	390.6	448
Rectangular trapezoidal	1.76	2.35	404.3	540

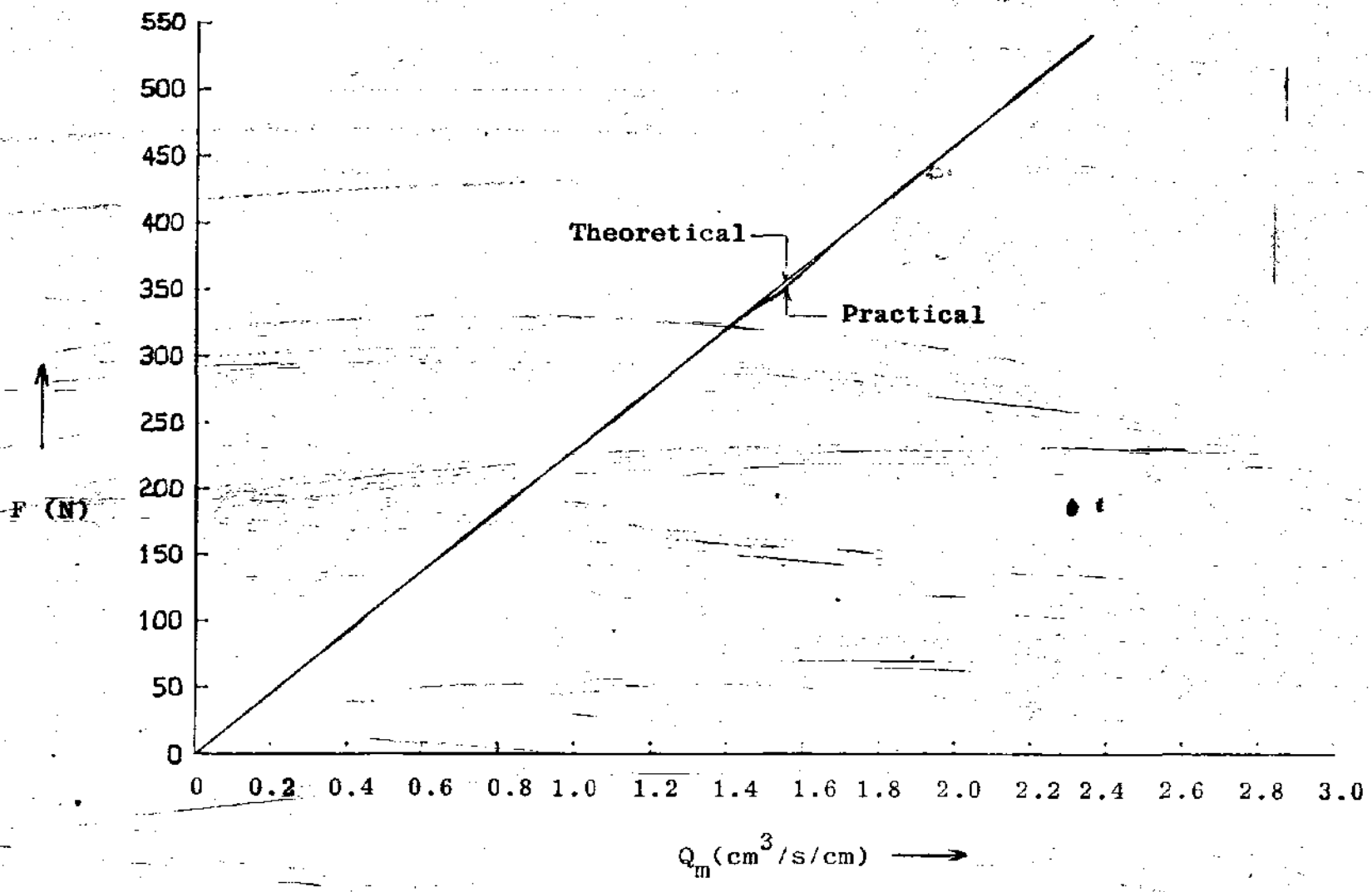


Fig.5.6 Variation of seepage force F with seepage discharge Q_m.

5.12 Variation of seepage discharge with blanket length

Certainly the length of the blanket influences its efficiency in reducing seepage and hence affects the quantity of seepage discharge. For this reason we will produce a graphical illustration of the behaviour of the discharge quantity with variation in the length of the blanket. Two shapes - the rectangular and triangular are investigated as it is hoped that the results produced by them would apply generally to others.

(i) Rectangular blanket

We recall the seepage equation given by equation (5.2)

$$Q_m = k_f bhT \{bL_d + \tanh(bL_1)\}^{-1}$$

which can be put in the form

$$Q_m = \frac{k_f bhT}{bL_d + \tanh(bL_1)} \qquad 5.37$$

$$= \frac{k_f bhT}{bL_d + 1 - \frac{2}{e^{2bL_1}+1}} \qquad 5.38$$

Denote $k_f bhT$ by C and $bL_d + 1$ by k, to get

$$Q_m = \frac{C}{k - \frac{2}{e^{2bL_1}+1}} \qquad 5.39$$

Using the experimental results in equation (5.39) gives

$$Q_m = \frac{0.09 \times 1.777 \times 10^{-3} \times 37 \times 38}{\{(1.777 \times 10^{-3} \times 40)+1\} - \frac{2}{e^{2bL_1}+1}}$$

$$= \frac{0.2248615}{1.07108 - \frac{2}{e^{3.554 \times 10^{-3} L_1}+1}} \qquad 5.40$$

Eq.(5.40) gives the general equation for the seepage discharge when all other quantities except L_1 are fixed.

TABLE 5.13 VARIATION OF THE SEEPAGE DISCHARGE WITH THE RECTANGULAR BLANKET LENGTH

L_{1cm}	0	10	20	30	40	50	60	70	80	90	100
Q_m	3.16	2.53	2.11	1.81	1.58	1.41	1.27	1.15	1.06	0.98	0.91

(ii) Triangular blanket

In carrying out similar calculations for the triangular blanket, we recall Eq.(5.4)

$$\frac{Q_m}{Q_o} = L_d\left[L_d + \frac{\sqrt{L_1}\, J_{1(\tau)}}{(\frac{\alpha}{S})^{\frac{1}{2}} J_{o(\tau)}}\right]^{-1} \qquad 5.41$$

$$\tau = 2\,(\frac{\alpha L_1}{S})^{\frac{1}{2}} \qquad 5.42$$

$$S = \frac{t}{L_1}$$

Substituting for S in Eq.(5.42) gives

$$\tau = 2\,(\frac{\alpha L_1^2}{t})^{\frac{1}{2}} = 2L_1(\frac{\alpha}{t})^{\frac{1}{2}} \qquad 5.43$$

We also recall that

$$J_{1(\tau)} = \frac{\tau}{2} - \frac{\tau^3}{2^3 1!2!} + \frac{\tau^5}{2^5\, 2!3!} - \cdots$$

and

$$J_{o(\tau)} = 1 - \frac{\tau^2}{2^2(1!)^2} + \frac{\tau^4}{2^4(2!)^2} - \cdots$$

Substitute for S, $J_{1(\tau)}$ and $J_{o(\tau)}$ in Eq.(5.41) to get

$$\frac{Q_m}{Q_o} = L_d\left[L_d + \frac{\sqrt{L_1}\left\{-\frac{2L_1(\frac{\alpha}{t})^{\frac{1}{2}}}{2} - \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^3}{2^3 1!2!} + \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^5}{2^5\ 2!\ 3!}\right\}}{(\frac{\alpha L_1}{t})^{\frac{1}{2}}\left(1 - \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^2}{2^2(1!)^2} + \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^4}{2^4(2!)^2}\right\}}\right]^{-1}$$

Substituting $\alpha = 3.15789 \times 10^{-5}$ and $t = 10$ as used in the experiment results in

$$\frac{Q_m}{Q_o} = L_d\left[L_d + \frac{\sqrt{L_1}\{1.777\times10^{-3}L_1 - 7.015\times10^{-10}L_1^3 + 1.477\times10^{-15}L_1^5\}}{1.777\times10^{-3}\{1-3.158\times10^{-6}L_1^2 + 2.493\times10^{-12}L_1^4\}}\right]$$

hence

$$Q_m = \frac{Q_o L_d}{L_d + \frac{\{1.777\times10^{-3}L_1 - 7.015\times10^{-10}L_1^3 + 1.4777\times10^{-15}L_1^5\}}{1.77\times10^{-3}\{1-1.58\times10^{-6}L_1^2 + 2.493\times10^{-12}L_1^4\}}} \qquad 5.44$$

TABLE 5.14 VARIATION OF THE SEEPAGE DISCHARGE WITH THE TRIANGULAR BLANKET LENGTH.

L_{cm}	0	10	20	30	40	50	60	70	80	90	100
Q_m ($cm^3/s/cm$)	3.16	2.53	2.11	1.80	1.58	1.40	1.26	1.14	1.04	0.96	0.88

5.13 Variation of seepage discharge with the blanket thickness

The thickness of the blanket t, like the length of the blanket plays a certain role in the blanket efficiency and the amount of seepage discharge. Analytical investigation of the relationship between Q_m and t is given here

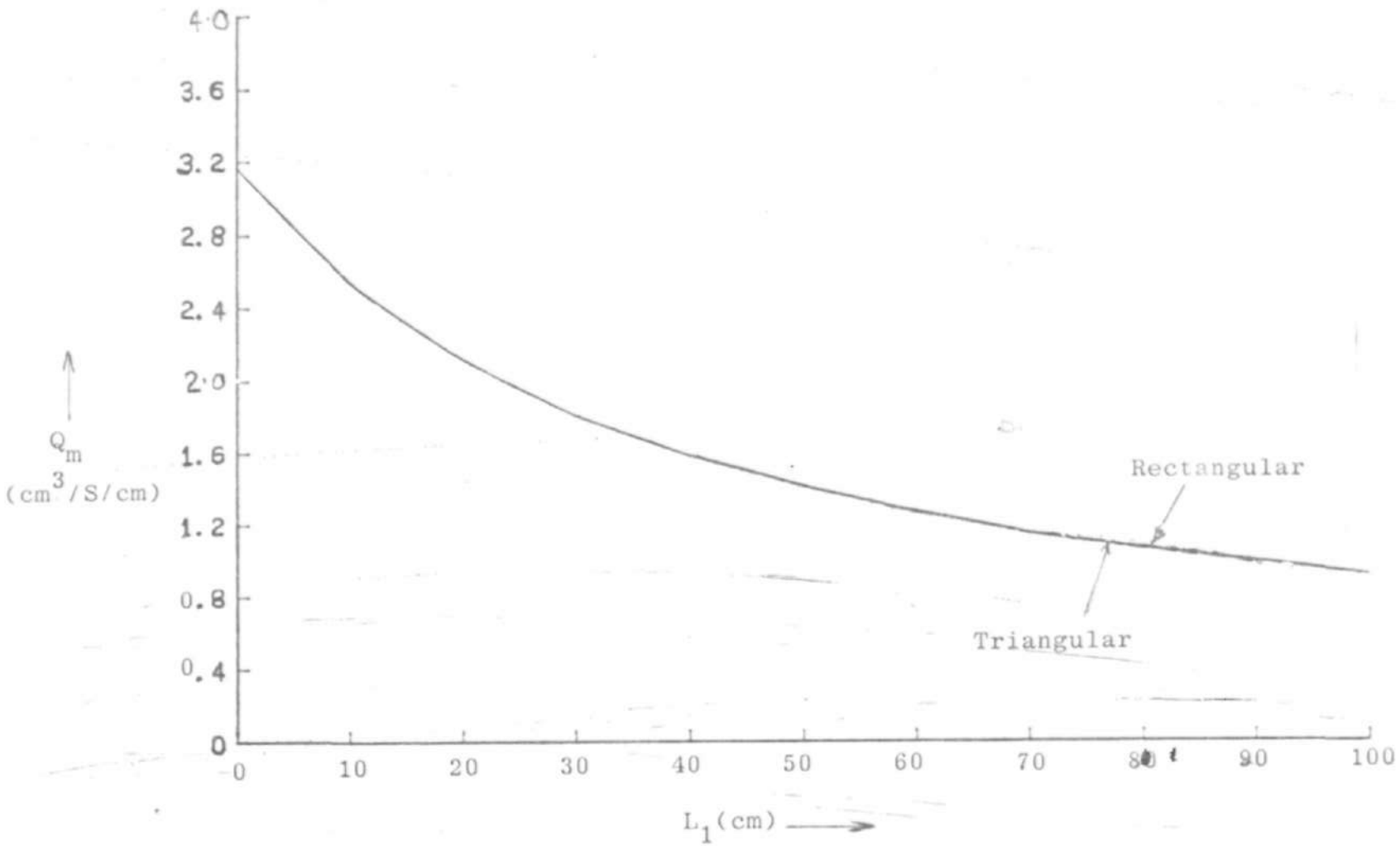


Fig.5.7 Variation of Q_m with L_1

for the rectangular and triangular blankets respectively.

(a) Rectangular blanket

We write down the seepage discharge quantity equation as

$$Q_m = k_f bhT \{bL_d + \tanh(bL_1)\}^{-1}$$

Substitute $(\frac{k_b}{k_f Tt})^{\frac{1}{2}}$ for b to get

$$Q_m = k_f(\frac{k_b}{k_f Tt})^{\frac{1}{2}} hT\{(\frac{k_b}{k_f Tt})^{\frac{1}{2}} L_d + \tanh[(\frac{k_b}{k_f Tt})^{\frac{1}{2}} L_1]\}^{-1} \qquad 5.45$$

Rearranging,

$$Q_m = h\frac{(k_b k_f)^{\frac{1}{2}}}{\sqrt{t}}\{\frac{(k_b)^{\frac{1}{2}}(k_f T)^{-\frac{1}{2}}}{\sqrt{t}} L_d + \tanh(\frac{k_b^{\frac{1}{2}}(k_f T)^{-\frac{1}{2}} L_1}{\sqrt{t}})\}^{-1}$$

Substituting h=37, k_b=1.08 x 10^{-4}, k_f=0.09 and T = 38, L_1=50 as used in the experiment we obtain

$$Q_m = \frac{0.71109}{\sqrt{t}}\{\frac{0.22478}{\sqrt{t}} + \tanh(\frac{0.28097}{\sqrt{t}})\}^{-1}$$

$$= \frac{0.71109/\sqrt{t}}{0.22478/\sqrt{t}+\tanh(0.28097/\sqrt{t})}$$

Multiply top and bottom by 0.3161062 and invert to get

$$\frac{1}{Q_m} = \frac{0.3161062\{0.22478/\sqrt{t} + \tanh(0.28097/\sqrt{t})\}}{0.3161062 \times 0.71109/\sqrt{t}}$$

$$= \frac{0.3161062\{0.22478/\sqrt{t} + \tanh(0.28097/\sqrt{t}\}}{0.22478/\sqrt{t}}$$

from which

$$\frac{1}{Q_m} = 0.3161062\{1 + \frac{\sqrt{t}\tanh(0.28097/\sqrt{t})}{0.22478}\}$$

which finally gives

$$\frac{1}{Q_m} = 0.3161062 \{1 + \frac{\sqrt{t}}{0.22478}(1 - \frac{2}{e^{2x0.28097/\sqrt{t}_{+1}}})\} \qquad 5.46$$

The beauty of this equation lies in the fact that it clearly shows that when t=0,

$$Q_m = \frac{1}{0.3161062} = 3.16 cm^3/s/cm$$

TABLE 5.15 VARIATION OF SEEPAGE QUANTITY WITH RECTANULAR BLANKET THICKNESS

t_{cm}	0	0.1	0.2	0.5	1	2	3	4	5	6	7
$Q_{cm^3/s/cm}$	3.16	1.582	1.500	1.446	1.426	1.416	1.413	1.411	1.410	1.409	1.408

(b) Triangular blanket

For the investigation of the influence of t on the seepage discharge under the triangular blanket, let us write down the previous equation of flow as

$$\frac{Q_m}{Q_o} = L_d[L_d + \frac{\sqrt{L_1}\ J_{1(\tau)}}{(\alpha/S)^{\frac{1}{2}} J_{o(\tau)}}]^{-1}$$

which on substituting $S = \frac{t}{L_1}$ gives

$$\frac{Q_m}{Q_o} = L_d[L_d + \frac{\sqrt{L_1}\ \sqrt{t}}{\sqrt{\alpha}\ \sqrt{L_1}} \frac{J_{1(\tau)}}{J_{o(\tau)}}]^{-1} \qquad 5.47$$

and

$$\tau = 2(\frac{\alpha L_1}{t/L_1})^{\frac{1}{2}} = 2L_1(\frac{\alpha}{t})^{\frac{1}{2}}$$

Using the Bessel's functions,

$$J_{1(t)} = \frac{2L_1(\frac{\alpha}{t})^{\frac{1}{2}}}{2} - \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^3}{2^3 1!\ 2!} + \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^5}{2^5 2!\ 3!} - \cdots$$

$$J_{o(\tau)} = 1 - \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^2}{2^2(1!)^2} + \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^4}{2^4(2!)^2} - \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^6}{2^6(3!)^2}$$

Substituting for $J_{1(\tau)}$ and $J_{o(\tau)}$ in Eq.(5.47) gives

$$\frac{Q_m}{Q_o} = L_d[L_d + \frac{t^{\frac{1}{2}}\{(\frac{2L_1(\frac{\alpha}{t})^{\frac{1}{2}}}{2}) - \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^3}{2^3 1!\ 2!} + \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^5}{2^5 2!\ 3!}\}}{\sqrt{\alpha}\{1 - \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^2}{2^2(1!)^2} + \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^4}{2^4(2!)^2} - \frac{[2L_1(\frac{\alpha}{t})^{\frac{1}{2}}]^6}{2^6(3!)^2}\}}]^{-1}$$

which upon simplification gives

$$\frac{Q_m}{Q_o} = L_d[L_d + \frac{t^{\frac{1}{2}}\{\frac{L_1\sqrt{\alpha}}{\sqrt{t}} - \frac{L_1^3\alpha^{3/2}}{2t^{3/2}} + \frac{L_1^5\alpha^{5/2}}{12t^{5/2}}\}}{\alpha^{\frac{1}{2}}\{1 - \frac{L_1^2\alpha}{t} + \frac{L_1^4\alpha^2}{4t^2} - \frac{L_1^6\alpha^3}{36t^3}\}}]^{-1}$$

Further simplification gives

$$\frac{Q_m}{Q_o} = L_d[L_d + \frac{L_1\alpha^{\frac{1}{2}} - \frac{L_1^3\alpha^{3/2}}{2t} + \frac{L_1^5\alpha^{5/2}}{12t^2}}{\alpha^{\frac{1}{2}} - \frac{L_1^2\alpha^{3/2}}{t} + \frac{L_1^4\alpha^{5/2}}{4t^2} - \frac{L_1^6\alpha^{7/2}}{36t^3}}]^{-1}$$

Substitute $L_d=40$, $L_1=50$ and $\alpha=3.15789 \times 10^{-5}$ to get

$$\frac{Q_m}{Q_o} = 40[40 + \frac{0.280975 - \frac{0.01109}{t} + \frac{1.45935 \times 10^{-4}}{t^2}}{5.61951\times10^{-3} - \frac{4.43644\times10^{-4}}{t} + \frac{8.75613\times10^{-6}}{t^2}}]^{-1}$$

which finally gives

$$\frac{Q_m}{Q_o} = 40[40 + \frac{0.280975t^3 - 0.01109t^2 + 1.45935x10^{-4}t}{5.61951t^3 - 4.43644t^2 + 8.75613x10^{-6}t}]^{-1}$$

It is easily seen from this expression that at t=0, when there is no blanket,

$$Q_m = Q_o$$

TABLE 5.16 VARIATION OF SEEPAGE QUANTITY WITH TRIANGULAR BLANKET THICKNESS

$t_{(cm)}$	0	0.1	0.2	0.5	1	2	3	4	5	6	7
$Q_{cm^3/s/cm}$	3.16	1.580	1.495	1.440	1.400	1.395	1.390	1.385	1.383	1.381	1.380

The graphical representation of this relationship is given in Figure 5.8.

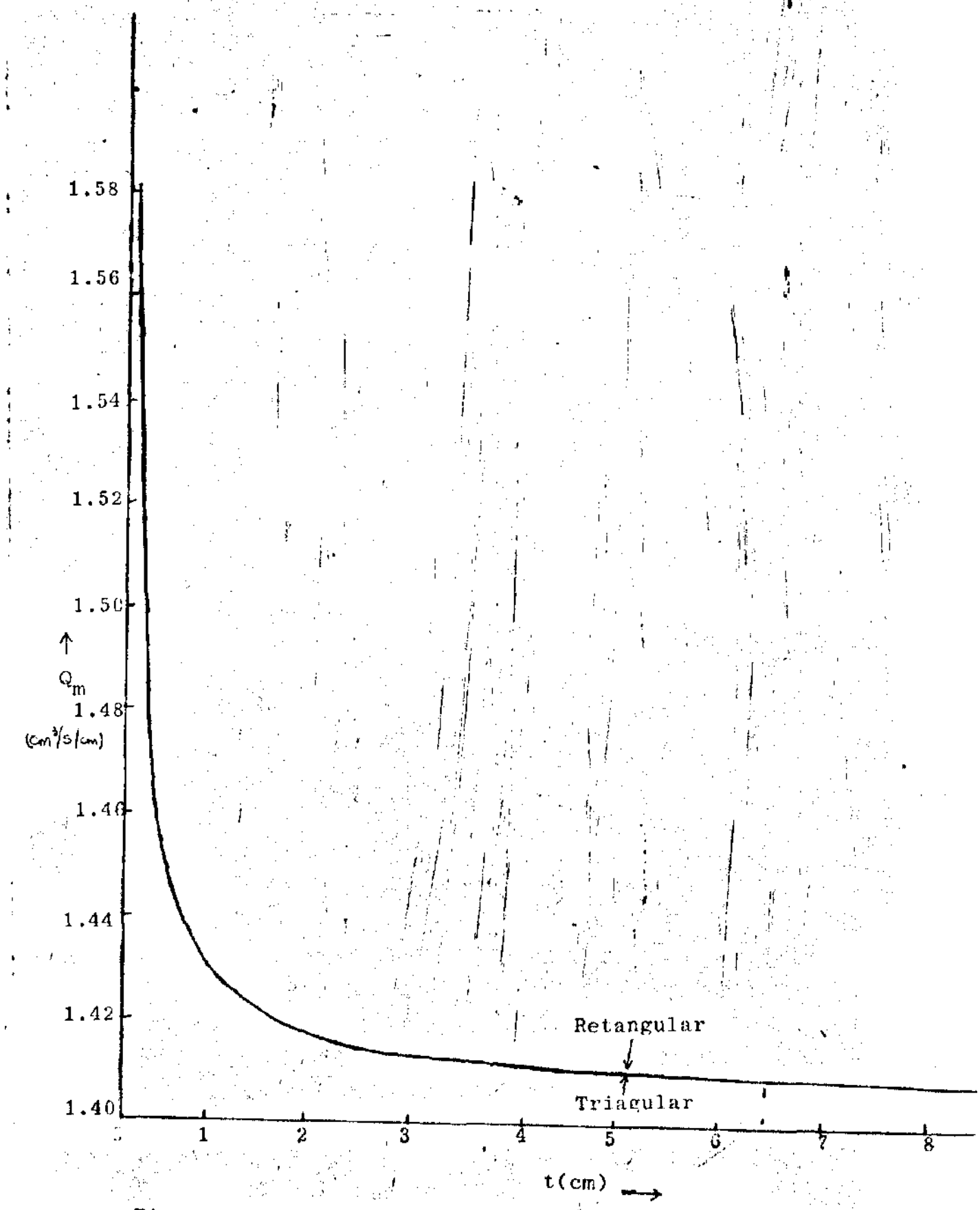


Figure 5.8 Variation of seepage discharge with the blanket thickness

CHAPTER 6

DISCUSSION OF THE RESULTS

6.1 Permeability coefficient

Although the coefficient of permeability is often considered to be a constant for a given soil (or rock), it can vary widely for a given material or medium, depending on a number of factors. Its absolute value depends, first of all on the properties of water and on the characteristics of the soil. Therefore, for practical purposes, all permeability values determined should be interpreted with caution.

During the experiments in this study, it was found that the effective permeability coefficients of the soil materials deviated slightly from those predetermined. While the practical permeability of the sand deviated from the theoretical value by 20 per cent, the difference between the theoretical and measured coefficients of the clay is 4.8 per cent.

The seepage coefficient differs because of the fluid in the soil, partially for its viscosity (which is the most important) and, consequently, its temperature. These conditions cannot be said to remain constant at the time of determination of permeability and during the experimental runs. Generally, to remedy this situation, it has been proposed by Polubarinova-Kochina (1962), that the seepage coefficient determined in one way or the other be reduced to a temperature of 0°C or 10°C. Then the coefficient of seepage for a temperature t°C is computed by a formula of

the form

$$k = k_o(1 + 0.03371t + 0.000221t^2) \qquad 6.1$$

due to Poiseuille

or

$$k = k_{10}(0.7 + 0.03t) \qquad 6.2$$

known as Hazen formula.

In the same vein the US Geological Survey has defined the laboratory (or standard) coefficient of permeability as that determined at 60°F.

Apart from the water factor, the soil composition, magnitude and form of its grains contribute to the variation in the theoretical and measured or practical values of the permeability coefficient. The size and continuity of the pore space or joints through which the fluid flows depend largely on

(a) the size and shape of the soil particles

(b) the density, and

(c) the detailed arrangement of the individual soil grains, or the structure.

There are also the problems of discontinuities (that is the presence of undetected joints or seams), and the presence of colloidal and inorganic materials. All these factors, most of which could normally have changed from the time of laboratory determination of k to the time of actual performance of the foundation and the blanket, led to the modification of the predetermined k. Moreover the inadequate compaction of the sand (in its dry state) and in the fragile tank in addition to possible over compaction

of clay led to variation in the permeability coefficients.

In summary, the permeability value determined in the laboratory did not give the exact value because of the difficulty of including all of the possible variations of the two soil materials. These include the porosities, packings and grain orientations, which were markedly changed as a result of disturbance and repacking of the unconsolidated materials.

6.2 Difference in theoretical and experimental results

Throughout the recorded readings disparities, though little, were found to exist between the calculated and experimental results. Prominent among the differing results are the underseepage discharges and the uplift forces thereby generated. The following reasons are advanced to account for the differences.

The assumption that the seepage flow is horizontal in the foundation is highly restrictive which might not exactly hold at many locations; as the actual velocities of the same magnitude have a downward vertical component (neglected in Darcy's formula) so that a greater saturated thickness is required for the same discharge. Another reason is that there could be a non-uniformity in the thickness of the foundation and the blanket. Obviously, it is practically difficult to level the top surfaces of the materials evenly. One other reason is the possibility of ununiform compaction of the blanket since a certain portion could be more compacted than the other. The result is the variation in permeability (the effect of which has already been noted)

across the top surface of the blanket. There is also the possibility of vertical flow occuring between the blanket and the wall of the tank leading to a greater and misleading seepage discharge.

It would be noted that for the composite shapes of the blanket, the experimental results are less than those evaluated theoretically. The only possible reason for this is that the compaction of the entire blanket must have been too high resulting in reduction of the permeability. This in turn would lead to much low flow as at the time of measuring the seepage discharge.

Less obvious is the truncation made in the application of Bessel's equations. These would create some difference depending on where the truncation was made.

6.3 The Efficiency of the Upstream Blanket

In all the experiments conducted, it was observed that an upstream blanket could help greatly in reducing the quantity of the underseepage discharge and the uplift force resulting from seepage. The theoretical and experimental results show that between 30% and 56% reduction in seepage discharge could be achieved with the application of an upstream blanket; (Table 6.2). The amount of reduction of the discharge could be increased when the ratio (k_f/k_b) is very large. Therefore, in practice where no seepage is desired, it is good practice to use a blanket material which is very nearly impervious (or possibly, totally impervious). It should be noted that high ratio of (k_f/k_b)

does not imply that the permeability of the foundation material, k_f be made high. It is obvious that low permeable foundations would help to obstruct the flow of seepage.

6.4 Relative efficiency of blanket shapes considered

Theoretical analyses and experimental investigations were made to determine which blanket shape would produce the greatest reduction in the quantity of seepage discharge and the corresponding uplift force; and hence the most efficient design. Graphical representations of the theoretical and experimental results were constructed to help to confirm the two sets of results. Figures 5.4 and 5.5 in particular have great significance in assessing the efficiency of a blanket in that the variable (on X-axis) in corporates all the properties (k_b, k_f) and the dimensions of the blanket shapes and the foundation. In all the shapes considered, the length L_1 and the height of the blanket at the impervious section of the dam together with the reservoir head were kept constant. Hence, the only parameter varied was the shape of the blankets so that reliable results or conclusions would be drawn from the camparison of their efficiency. Given below is a tabular comparison of the performance of the blanket shapes.

TABLE 6.1 THEORETICAL AND EXPERIMENTAL DISCHARGES

Shape of blanket	Discharge (cm^3/s)	
	*Theoretical	**Experimental
Rectangular	1.41	1.60
Triangular	1.40	1.45
Trapezoidal	1.48	1.55
Stepped-rectangular	1.95	1.70
Rectangular-trapezoidal	2.35	1.76

*Results obtained for fixed volume
**Results obtained for fixed length

To pass judgement on the relative effectiveness of each blanket, we will first of all evaluate the percentage reduction both in theoretical and experimental results for each of the blanket shapes.

Rectangular blanket

For this type of blanket design, the theoretical seepage discharge calculated from equation (5.2)

$$= 1.41 cm^3/s/cm$$

Seepage discharge determined from experiment

$$= 1.60 cm^3/s/cm$$

The experimental seepage discharge in the absence of any blanket

$$3.16 cm^3/s/cm$$

Therefore theoretical seepage reduction

$$= \frac{3.16-1.41}{3.16} \times \frac{100}{1}$$

$$= 55\%$$

Practical seepage reduction,

$$= \frac{3.16 - 1.60}{3.16} \times \frac{100}{1}$$

$$= 49\%$$

Triangular blanket

From Table 6.1, the theoretical seepage discharge for this blanket design

$$1.40 cm^3/s/cm$$

The experimental seepage discharge

$$= 1.45\ cm^3/s/cm$$

Hence the theoretical seepage reduction

$$= \frac{3.16 - 1.40}{3.16} \times \frac{100}{1}$$

$$= 56\%$$

and practical seepage reduction

$$= \frac{3.16 - 1.45}{3.16} \times \frac{100}{1}$$

$$= 54\%$$

Trapezoidal blanket

For this type of blanket design, the theoretical seepage discharge calculated from equation (5.9)

$$= 1.48 cm^3/s/cm$$

The practical seepage discharge determined experimentally

$$= 1.55 cm^3/s/cm$$

The theoretical seepage reduction

$$= \frac{3.16 - 1.48}{3.16} \times \frac{100}{1}$$

$$= 53\%$$

The practical seepage reduction

$$= \frac{3.16 - 1.55}{3.16} \times \frac{100}{1}$$

$$= 51\%$$

Stepped-rectangular blanket

From equation (5.12), the theoretical seepage discharge for this type of blanket design

$$= 1.95 \text{ cm}^3/\text{s/cm}$$

From Table (6.1), the experimental seepage discharge

$$= 1.70 \text{cm}^3/\text{s/cm}$$

Hence, the theoretical seepage reduction

$$= \frac{3.16 - 1.95}{3.16} \times \frac{100}{1}$$

$$= 38\%$$

The practical seepage reduction

$$= \frac{3.16 - 1.70}{3.16} \times \frac{100}{1}$$

$$= 46\%$$

Trapezoidal-rectangular blanket

For this type of blanket design, the theoretical seepage discharge calculated from equation (5.14)

$$= 2.35 \text{cm}^3/\text{s/cm}$$

The practical discharge as determined from experiment

$$= 1.76 \text{cm}^3/\text{s/cm}$$

The theoretical seepage reduction

$$= \frac{3.16 - 2.35}{3.16} \times \frac{100}{1}$$

$$= 26\%$$

The practical seepage reduction

$$= \frac{3.16 - 1.76}{3.16} \times \frac{100}{1}$$

$$= 44\%$$

The above results are tabulated below for easy comparison of the efficiency of the various blanket shapes.

TABLE 6.2 EFFICIENCY OF THE BLANKET DESIGNS

Shape of Blanket	Seepage reduction (%)	
	*Theoretical	**Practical
Rectangular	55	49
Triangular	56	54
Trapezoidal	53	51
Stepped-rectangular	38	46
Rectangular trapezoidal	26	44

*Results obtained for fixed volume
**Results obtained for fixed length

From the above table, it is seen that the triangular shaped blanket design has the greatest efficiency in reducing underseepage. The rectangular and trapezoidal shaped blanket designs tend to have equal efficiency. The composite shapes have the least efficiency though the stepped-rectangular shaped blanket is to be preferred.

For the final judgement, we will observe the graphical illustrations in Figures 5.4 and 5.5, the calculations for which incorporated all the dimensional features and the properties of the two soil materials. From Figure 5.4 which is the theoretical representation of the optimal performance of the blanket designs, we have that the triangular blanket records the lowest ratio (Q_m/Q_o) for a given value of $L_d(\frac{\alpha}{V})^{\frac{1}{3}}$. Second to the triangular blanket is the trapezoidal shaped blanket design while the rectangular, the stepped-rectangular and the rectangular-trapezoidal shaped blanket designs follow in that order. From the plot of the experimental results Figure 5.5, the result is the

same as regards efficiency. In general, therefore, we can conclude with a measure of confidence that the triangular shaped blanket design has the greatest efficiency in reducing underseepage discharge and subsequently the uplift pressures. The trapezoidal shaped blanket design follows and then the rectangular shaped blanket design. The composite shaped blankets performed least in efficiency, nonetheless, the stepped-rectangular shaped blanket is more effective than the rectangular-trapezoidal blanket.

The following reasons are proposed for the above findings from the study. That the triangular shaped blanket design is more efficient than any other blanket design shape lies wholly on its slope. With reference to Figure 5.7 (which also proves that triangular blanket shape reduces underseepage move than the rectangular shaped blanket), we see that Q_m remains constant with increase in length of blanket for a given thickness, t. But as soon as L_1 is increased, keeping the thickness t constant, the slope is decreased and there occurs a great reduction in Q_m which is greater than that in the rectangular blanket design shape with the same length, Fig.5.7. Further increase in the length will reveal more pronounced reduction in seepage than the rectangular blanket shape does.

If on the other hand, the thickness at the upstream section, t is increased, the slope of the triangular shaped blanket is increased. The corresponding decrease in seepage discharge is much lower than that recorded in the rectangular blanket shape for the same additional layer. However, the

net reduction in seepage for any given blanket thickness is still greater in the triangular shaped blanket. This is because the initial thickness is probably effectively functional in the process of seepage reduction, Figure 5.8.

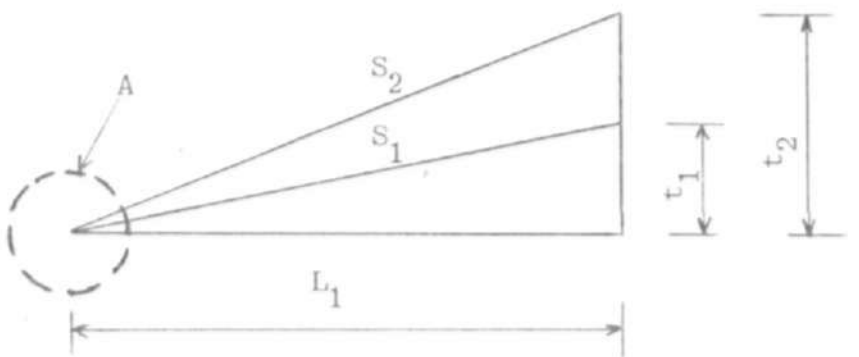


Figure 6.1 Slopes of a triangular shaped blanket

Hence subsequent rise in the thickness, t appears to help only in blocking undetected pores in the previous layer (S_1). Also it would prevent the end of the triangle (portion A) from being washed probably due to eddies of incoming water during reservoir filling. There is the possibility that the rising slope of the blanket toward the impervious section of the dam tends to pose difficulty to the seeping water which increases toward the dam.

There is head loss due to resistance of the blanket slope and increase in seepage line.

That the trapezoidal shaped blanket design performs better than the rectangular shaped blanket should not be far fetched. Generally, the smaller of the parallel sides of the trapezium is much less than the larger side. Then due to the action of water, the shape is gradually reduced or transformed into a triangular shape. So the trapezoidal shaped

blanket will then act or function like a triangular shaped blanket.

It should be mentioned that the volume of soil material used in each blanket shape design differs. This was because investigating into the shape of the blanket design for effectiveness required that the basic dimensions be kept constant. So, the length and thickness of the blanket shapes at the upstream section of the dam remained the same in all cases. From the definition sketch of the rectangular shaped blanket given in Fig.(4.4) the volume of material used for it = 50 x 50 x 10 = 25,000cm^3.

From Fig.(4.5), the volume of material used for constructing the triangular shaped blanket design = $\frac{1}{2}$(10x50)x50 = 12,500cm^3

In constructing the trapezoidal shaped blanket design, the volume of clay used (Fig.4.6), = $\frac{1}{2}$(10+1.8)x50x50 = 14,750cm^3.

For the stepped-rectangular blanket design, the volume of the clay material used = 50(25x10 + 25x5)

= 18750 cm^3 (Fig.4.7).

From Fig.(4.8), the volume of material used in constructing the rectangular-trapezoidal blanket design

= 10x10x50 + $\frac{1}{2}$(10+2)x40x50

= 17,000 cm^3

6.5 Applicability of the Results

For practical purposes, with view to conserving water and preventing dam failure, the dam designer would find the results of this study highly invaluable. He would be able to decide on the shape of the blanket to be designed

for his project given the construction facilities. Depending on the amount of reduction of seepage discharge desired he would only have to choose from among the formulae contained herein to calculate the parameters required. It has already been noted that subsequent increase in the thickness of blanket is negligibly functional in its efficiency.

For theoretical application, when a similar experiment is to be performed, one would be able to decide on the dimensions of the sand model to suit his objectives. In addition, the mathematical formulae provided would aid the researcher to determine all the desired seepage quantities.

Moreover, it provides a useful tool or guidelines to the researcher. The theory of seepage consists of a wide field of study. Therefore he would find this work as a useful fundamental companion which he could extend or develop depending on the field of his research.

It is important to note that Q_o as used herein is only of theoretical significance. Therefore, in practice the anticipated actual Q_o obtained from k_f, h, T and L_d should be used to determine the thickness and length of the blanket.

CHAPTER 7

SUMMARY, RECOMMENDATIONS AND CONCLUSION

7.1 Summary

The effectiveness of upstream blanket designs in controlling underseepage and their relative performance have been studied. The concepts derived in the study have introduced more clarity and complimented the past investigations of a series of scientists.

Experiments were conducted in the laboratory using a seepage tank model constructed of 6mm thick glass perspex sheets. In the model, the foundation was made of a mixture of medium and coarse sand. Clay was used to build the respective blanket design shapes. The blanket design shapes used in the experiments consisted of:

1) rectangular
2) triangular
3) trapezoidal
4) stepped-rectangular which consisted of two rectangular sections with equal lengths but different thickness and
5) rectangular-trapezoidal which had a rectangular section joined to a trapezoidal section.

To prevent local scour of the blanket due to eddies at the time of filling the reservoir, a perforated plastic perspex sheet and wire gauze were used to cover the top surface of the blanket.

Under each blanket design shape, 10 different measurements of the underseepage discharge were taken and the

average determined. The average values were then compared to see which design shape gave the least discharge. Mathematical equations were developed to enable comparison between the theoretical and experimental results. Equations were also formulated to determine the variation of seepage discharge with the length and thickness of the blanket shapes respectively. Lastly, graphical illustrations were constructed to furnish logical conclusion on the relative effectiveness of the various upstream blanket design shapes.

7.2 Recommendations

7.2.1 Permeability

For a reliable theoretical consideration a more rational concept of permeability should be expressed by a coefficient which is independent of those fluid properties governing the flow. These include the viscosity and the specific weight. For the medium, the flow should be related to a pore diameter which can be assumed proportionate to a representative grain diameter. As a practical matter, materials commonly used in dam construction, are not completely impermeable, and foundation soils show wide ranges in primary and secondary permeability. Therefore, adequate precautionary measures should be taken to control the permeability of materials emplaced during construction. This is much simpler than reduction of undesirable permeability to a safe level beneath or in the vicinity of a dam. Arising from the very significance of permeability in dam building, we present here the do nots in permeability:

1. Do not design or build hydraulic structures without adequate permeability investigations.
2. Do not depend on novices to perform or interprete permeability studies; give this work to experts who understand earth deposits and know their limitations.
3. Do not underestimate the importance of minor soils or geologic details on the permeability of soil and rock formations. They cause a majority of failures of dams, reservoirs, and other hydraulic structures.
4. Do not allow low compaction of "permeable" drainage aggregates or embankment materials as a means of obtaining high permeability. Loosely compacted soils and aggregates that become saturated are likely to liquefy under severe shocks.

7.2:2 Deterioration of Foundations

Field exploration and geologic mapping for earth-dam projects, should be sufficiently thorough to disclose important soil and rock formations. It is important that any seepage through abutment or the foundation soil should be monitored, both as regards quantity and chemical composition. If seepage quantities increase for no apparent reasons, or if other unexplained changes in seepage conditions develop internal piping or heave may be indicated. The consequence is increase in hydrostatic pressure within the foundation and or removal of material by erosion that might cause structural instability. Immediate action should be taken by placing weighted filters over the zone of egress covered then by layers of rockfill. Whether

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further action is necessary will depend on the circumstances, particularly the possibility of build-up of high hydrostatic pressure in the foundation. If this is likely, then grouting of the foundation may be necessary and or the provision of designed drainage well that can remove seepage water safely - without the transport of foundation material.

If reservoir water can enter the rock formation then uplift forces will act on the dam. Apart from this, considerable jacking forces can accumulate - sufficient to dislodge rock masses from the canyon walls, and weaken the abutments. The normal treatment is to provide a grout curtain extending below the dam, along its axis or the extensions thereof into the abutments. Downstream from this, drainage holes can be provided and monitored.

7.2:3 Blanket Design

Feasibility of construction should govern the dimensions of a blanket as long as the extremes of inefficiency represented by the ends of end effects are avoided. Before accepting a blanket design as adequate, an investigation should be made to determine the probable degree of stratification of the foundation. Natural soil deposits are always more or less stratified and even moderate stratification can have an important influence upon lift pressure.

The study presented in this thesis did not consider the presence of tailwater. This apart, there are still

very many different shapes of blanket designs into which investigations can be conducted as to their efficiency in preventing or reducing underseepage. Such different composite shapes of upstream blanket into which future investigations are recommended are presented below.

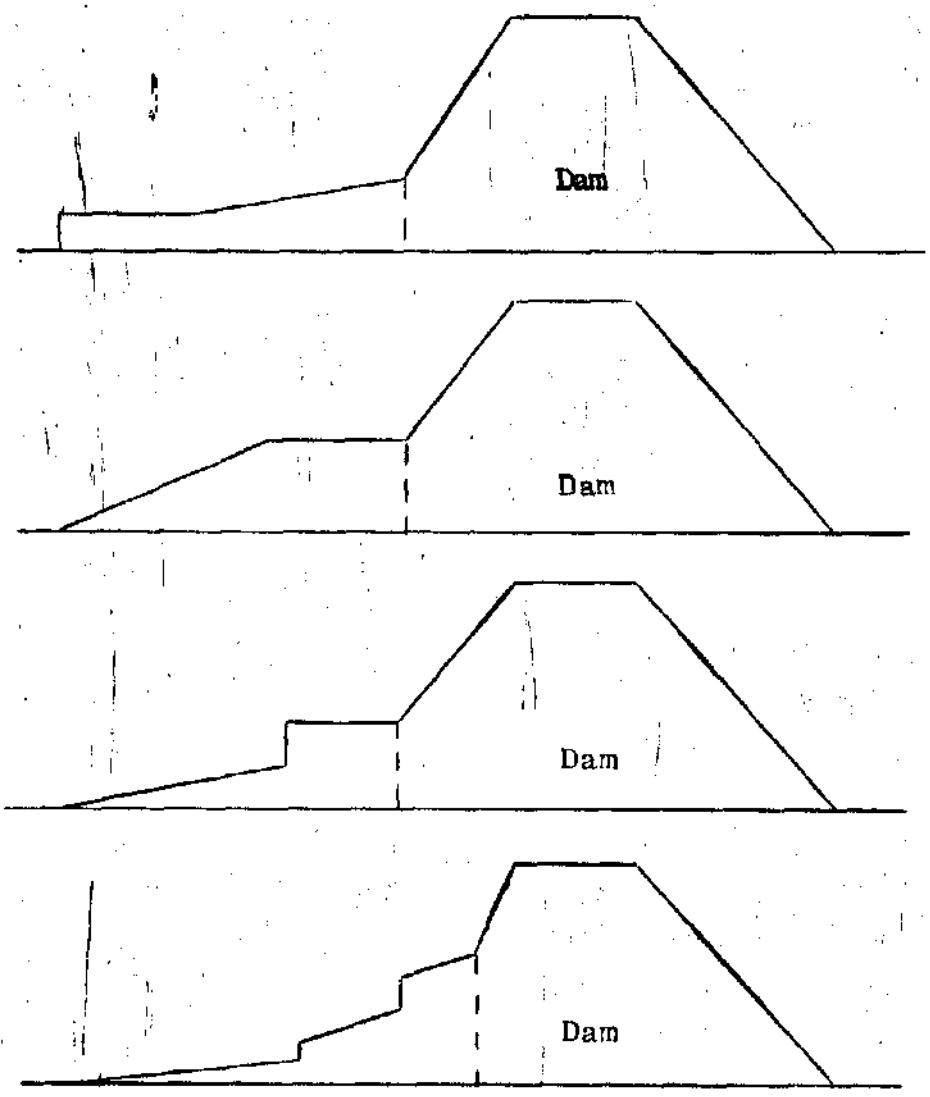


Fig.7.1 Possible upstream blanket designs.

A discussion of downstream drainage facilities falls outside the scope of this study. However, a knowledge of the downstream resistance is necessary in evaluating the effectiveness of a blanket design.

7.3 Conclusion

7.3:1 Safety of Dams

There are no accurate records of the number of dams that have failed through the history of their construction. The cost in human life, goods and property damage of a flood generated by breaching of a dam depends to a large extent on the magnitude of the flood and what lies in its pathway. More pertinent is the volume of water in the reservoir behind the dam and the configuration of the valley below it. Middlebrook's list of earth-dam failures presented in Appendix A, shows that a greater number have resulted from seepage or piping. Unfortunately, the world population is expanding daily with an increasing occupation of flood plains by dwellings, commercial and industrial facilities, and high ways. Therefore, dams built long ago and in recent times present a growing potential for massive destruction of life and property.

It is against this background that the responsibility for the safety of dams and reservoirs no longer belongs only to the designer and builder. It is shared by those who have knowledge and understanding, however imperfect they may be, of the expected behaviour of natural materials under the conditions superimposed by the loads of dams and reservoirs. Modern technology and special responsibility require that safe construction and maintenance of dams and

reservoirs shall be the shared responsibility of engineers, geologists, and rock mechanics experts. The engineers have developed the theory of static and dynamic stress and strain relationships in dams to a high state of sophistication. Given a uniform foundation of competent materials, they can predict the nature of the interaction between the dam and the natural materials below it and at its sides so as to allow for a large margin of safety. Recognition of the need for world-wide surveillance of dams and reservoirs with emphasis on their safety has resulted in the formation of the International Commission on Large Dams (ICOLD), a unit of the World Power Conference. Within individual countries increasing efforts are being made to regulate and maintain continued safety of dams and reservoirs through the close cooperation of engineering organizations and government agencies. In spite of these good works, dams continue to fail.

Consequently, earth-dam designers should not allow themselves to be deluded into believing that seepage will be automatically controlled if some kind of seepage control is specified. Controlling underseepage will always present challenges to even the most astute and experienced designers. This follows because water will always find the paths of least resistance through pervious strata, joints or seams as they really exist, rather than as they are assumed to be for purposes of analysis. For this reason, every dam should be instrumented with piezometers, observation wells, and seepage measuring devices.

Adequate instrumentation and observation of completed earth dams and the correction of any unsafe or questionable condition is an important part of the overall design process. Having studied the efficiency of the respective blanket design shapes considered, the following conclusions are reached.

7.3:2 Effectiveness of Upstream Blanket

As in any other phases of engineering experiments, the validity of any conclusion based on the findings of this research is limited by the degree to which those idealized conditions and assumptions are approached in nature or experimentally. Likewise is the degree of accuracy obtainable in the determination of the permeability of the soils used. Nonetheless, the resulting analytical evaluation of a blanket design shape should be of value as an indication of the probable limits of its effectiveness. In concluding, it is pertinent to observe the net reduction of underseepage by the respective blanket design shapes given in the table 6.2.

From the results obtained we conclude that:

(1) There exists a limiting value for both the length and thinkness of blanket after which negligible reduction of underseepage is achieved.

(2) An upstream blanket can be very effective in reducing the underseepage discharge quantity and hence the uplift force.

(3) The triangular shaped blanket design is most effective in controlling or reducing underseepage and uplift pressure. The trapezoidal blanket design follows and then the rectangular blanket design as regards effectiveness. In that decreasing order, we have the composite shapes of blanket design. The results show that the stepped-rectangular blanket design is to be preferred to the rectangular-trapezoidal blanket shape as obtained theoretically.

(4) The force is reduced in the same proportion as the underseepage discharge in each of the blanket designs.

(5) Triangular shaped blanket has certain practical limits that should be considered. For instance, it is doubtful that a blanket with a featheredge would either be desirable or practical to construct.

(6) If the quantity of the blanket material as used for rectangular blanket is fixed, or $(\forall/\alpha)$ is fixed, for other design shapes, their dimensions will increase. Hence, for optimal conditions, Eq.(3.56) which reads $(L_1)_T = 1.193(L_1)_R$ finds a useful application. It implies that the length of the triangular blanket shall be about 19% greater than the length of the rectangular blanket.

(7) If, on the other hand, (Q_o/Q_m) is fixed for all the shapes, analysis reveals that $(L_1)_T = 1.055(L_1)_R$ as given in equation (3.60).

(8) Therefore, either of the three sections rectangular, triangular and trapezoidal may be used with very high effectiveness. Also for any shape of the blanket, a

considerable departure could be made from the theoretical optimum dimensions but without great loss of effectivenses.

(9) The experimental results which show that the triangular blanket is more effective than the rectangular blanket is not conclusive due to the fact that the volume of the material was not fixed. The lesser amount of seepage discharge in the triangular blanket could be due to over-compaction of the blanket material.

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APPENDIX A

MIDDLEBROOK'S COMPREHENSIVE LIST OF EARTH DAM FAILURES RESULTING FROM SEEPAGE AND OR PIPING

	Dam and Location	Date Built	Date Failed	Reference*	Height(m)	Type	Reason for unsatisfactory performance
1	Anacond, Mont	1898	1938	ENR Vol.121	21.95	Earth and concrete core	seepage slide
2	Apishapa, Colorado	1920	1923	ENR, Vol.91	35.05	Earth, rolled	Piping through settlement cracks
3	Ashti, India		1883	ASCE Proc. Vol.49	17.7	Earth rolled	seepage through foundation
4	Barton, Idahosa	1910	1922	Sherard	12	Earth rolled	Seepage slide
5	Blartown, Wyoming		1888	EN, Vol.47		Earth rolled	Piping along outlet
6	Bradford, England		1896	ASCE Proc., Vol.49	22	Earth rolled	Piping along outlet
7	Brooklyn, N.Y.	1893	1893	EN, Vol.47			Foundation seepage
8	Calaveras, Calif		1918	ENR, Vol.80	73	Hydraulic	Uplift pressure
9	Cold Springs, Colorado		1912	ER, Vol.66	15	Earth rolled	Embankment seepage
10	Consholocken Hill Pa.		1873	ENR, Vol.18		Earth rolled	Piping

11	Costiha, N. Mexico		1924 1941	ER, Vol.75	38	Earth rolled	Embankment seepage sloughs
12	Crane Creek, Idaho	1910	1928	Sherard	19	Earth and Puddle cone	Piping into Tunnel outlet
13	Dalton, N.Y.		1912	EN, Vol.67	9	Earth & concrete cone	Foundation piping
14	Davis Reservoir Calif.		1914	EN, Vol.72	12	Earth rolled	Piping around outlet
15	Desabia Forebay, Calif.	1903	1932	Sherard	16		Piping through embankment
16	Dry Creek, Mont	1938	1939	ENR, Vol.122	14	Earth, rolled	Piping
17	East Liverpool Ohio	1901	1901	ER Vol.44		"	Piping along outlet
18	English, Calif		1883	Schuyler	9	"	Piping
19	Frazier, Idaho	1915	1935	Sherard	8	"	Seepage slide
20	Fruit Growers Reservior, Colorado	1898	1937	ENR, Vol.118	11	"	" "
21	Greenlick, Pa. (Pensylvania)	1901	1904	EN, Vol.52	18	"	Seepage
22	Gunnison, Calif.		1890	EPG Journal, Vol.44	6	"	Piping along outlet
23	Hatchtown, Utah	1908	1914	EN, Vol.75	20	"	Seepage along conduit

24	Hebron, N. Mex.	1913	1914	ER Vol.69	17	Earth rolled	Piping through dam
25	Hope Reservioir, R.I.	1882	1907	ER, Vol.56	7	"	Seepage
26	Hornell, N.Y.	1912	1912	EN, Vol.58		Earth & concrete core	Seepage
27	Horse Creek, Colo.	1911	1914	EN, Vol.71	17	Earth rolled	Piping
28	Knoxville Reservoir Tenn.		1883	EN, Vol.47		"	Foundation seepage
29	La Fruta, Tex	1930	1930	ENR, Vol.106	18	"	Foundation piping
30	Lake Avalon, N. Mex.		1894	EN, Vol.54	14.5	"	Seepage
31	Lake Francis, Calif.	1899	1899	ASCE Trans., Vol.58	15	"	Piping along outlet
32	Lake George, Colo		1914	ASCE Proc., Vol.49		"	Piping
33	Lake Toxaway N.C.	1902	1916	ER, Vol.74	18.7	"	Seepage
34	Lancaster, Pa.		1894	ER, Vol.39	6.3	"	Piping along outlet
35	Lebanon, Pa.		1893	ER, Vol.27	12	"	Piping between fill and foundation
36	Long Tom, Idaho	1906	1915	Sherard	15	Earth and puddle core	Piping into tunnel

37	Longwalds Pond, Mass		1922	EBR, Vol.89	9	Earth and puddle core	Piping
38	Lyman, Arizona	1913	1915	EN, Vol.73	19.7	Earth rolled	Piping
39	Lynde Brook, Mass		1876	EPG Journal, Vol.44	8	" "	Piping along outlet
40	Magic, Idaho	1910	1911	ER, Vol.60	39	" "	" " "
41	Mahonoy City, Pa.		1892	EN, Vol.27		" "	Piping
42	Marshall Lake, Colo	1908	1909	ER, Vol.62	21		Seepage
43	Mellville, Utah	1907	1909		10.8	" "	Piping through foundation
44	Mill River, Mass	1865	1874	ASCE Trans., Vol.3	13	Earth and concrete core	Seepage
45	Mohawk, Ohio		1915	EN, Vol.73	5.4	Earth rolled	"
46	Mon treal, Que Canada		1896	EN, Vol.47	5.4	" "	"
47	Mud Pont, Mass	1873	1886	ER, Vol.13	4.5	" "	Piping
48	Nebraska City Nebr.	1890	1890	EN, Vol.47	5	" "	Seepage
49	New Bedford, Mass	1866	1868	ASCE Trans., Vol.2	7.5	" "	Piping along outlet condu
50	Pleasant Valley, Utah		1928	ENR, Vol.100	19	Earth & rock	Piping through settlement cracks
51	Porland, Me.	1889	1893	ER, Vol.28	13.5	Earth rolled	Piping along drain pipe

52	Rovborough Reservoir, Pa.	1894	1894	ER, Vol.30		Earth rolled	Piping
53	Scottdale, Pa.		1904	EN, Vol.52	20	" "	"
54	Shelton, Conn.		1903	ER, Vol.47	6	Earth & rock	"
55	Sinker Creek, Idaho	1910	1943	Sherard	21	Hydraulic & earth	Seepage slide
56	Spring Lake, R.I.	1887	1889	EN, Vol.20	5.4	Earth & rock	Piping along outlet
57	Staffordville, Conn.		1887	EN, Vol.4	6	" "	" " "
58	Swansea, Wales	1867	1879	SE, Vol.3	24	" "	Piping
59	Tupper Lake, N.Y.	1906	1906	EN, Vol.57	5.4	Earth rolled	Piping along outlet
60	Wersse, Czechoslovakia		1916		12.6	" "	" " "
61	West Julesburg, Colo	1905	1910	ER, Vol.63	16.5	" "	Piping
62	Wilmington, Del.	1887	1900	ER, Vol.42	3.6	" "	Piping along outlet
63	Wister, Okla.	1951		— —	27	" "	Piping
64	Yuba, Calif.	1949	1951	Sherard	7.5	" "	Seepage slide
65	Zuni, N.Mex.	1907	1909	EN, Vol.62	21	Earth & rock	Piping through abutment

ENR = Eng'g News Record
ER = Eng'g Record
SE = Sanitary Engr.
ASCE Proc. = ASCE Proceedings
ASCE Trans = ASCE Transactions

Sherard = "Influence of Soil Properties & Construction Methods on Performance of Homogeneous Earth Dams", by J.L. Sherard, 1952