FILTRATION PROPERTIES OF GEOTEXTILES

Thesis by

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To my wife TÜLİN

and my daughter EZGİ

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Emin MERDİN

A B S T R A C T

FILTRATION PROPERTIES OF GEOTEXTILES

In the past years,many models,and laboratory techniques and apparatus have been developed to examine the filtration properties of the geotextiles.To select the proper geotextile which will be used in drainage and filtration,the indicative pore size of the geotextile and granulometric curve of the soil must be determined at the begining.

The objective of this thesis is to construct the (Saturated Silty Soil + Geotextile + Wet Gravel Filter)system and, to determine the coefficient of permeability of this system.These tests are performed to determine when the clogging happens.For this reason,two types of silty soil,and four types of nonwoven geotextiles have been used.California Bearing Ratio (CBR) test molds have been used as test apparatus. Into the lower part of the mold,gravel filter has been placed, and geotextile has been placed between the upper and lower part of the mold,and saturated silty soil has been filled into the upper part of the mold.The coefficient of permeability of this system has been determined, and Number of Test versus system permeability curves have been drawn.

The test results have shown that,the coefficient of permeability of the system remains approximately constant after a definite time.

Ö Z E T

GEOTEKSTİLLERİN FİLTRASYON ÖZELLİKLERİ

Geçmiş yıllarda,geotekstillerin filtrasyon ve drenaj işlerinde davranışlarına ilişkin birçok modeller,laboratuar teknik ve aletleri geliştirilmiştir.Filtrasyon ve drenaj işlerinde kullanılacak uygun bir geotekstil seçiminin yapılabilmesi için ,geotekstilin delik dağılım eğrisi ile zeminin dane dağılım eğrisi ve permeabilitesi öncelikle belirlenmelidir.

Bu tezin konusu (Siltli zemin + Geotekstil + Islak Çakıl Filtre) sistemi oluşturularak bu sistemin geçirgenlik katsayısının bulunması ve tıkanmanın oluşmasının belirlenmesidir.Bu amaçla iki değişik siltli zemin ve dört değişik tip örgüsüz geotekstil kullanılmıştır.Bu sistem California Bearing Ratio (CBR) Test kalıbının kullanılması ile kurulmuştur.Kalıbın alt parçasına çakıl,iki kalıbın arasına geotekstil ve kalıbın üst parçasınada suya doygun siltli zemin koyularak sistemin geçirgenliği belirlenmiştir.Tıkanmanın oluşmasını görmek için deney sayısına karşı geçirgenlik eğrileri çizilmiştir.

Deney sonuçları göstermiştir ki: zamanla geotekstildeki tıkanma yaklaşık olarak sabit kalır.

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A = Area of tube, Coefficient for the soil confining effect of fabric, area of the cylinder, cross-sectional area of soil sample.

a = The radius of loaded area, area of standpipe.

B = A function of the uniformity coefficient of soil.

c = The shear strength of soil.

C_1 = Factor varying with the size of casing.

C_p = Factor varying with the size of the test hole, and the length of test section.

$D_{15\ filter}$ = 15% size of filter material.

$D_{15\ soil}$ = Soil diameter below which lie 15% of soil particles.

$D_{50\ soil}$ = Soil diameter below which lie 50% of soil particles.

$D_{85\ soil}$ = Soil diameter below which lie 85% of soil particles.

$D_{90\ soil}$ = Soil diameter below which lie 90% of soil particles.

D_h = Hole diameter.

d = Estimated rut depth, depth of hole below water table.

EOS_{fabric} = Equivalent Opening Size of fabric.

GR = Gradient Ratio.

G_s = Specific Gravity of soil.

H = The longest drainage path.

h = The net hydrostatic head.

h_1 = Hydraulic head across sample at the beginning of test.

h_2 = Hydraulic head across sample at the end of the test.

i = Hydraulic gradient.

k = Coefficient of permeability.

k_{fabric} = Coefficient of permeability of fabric.

k_{soil} = Coefficient of permeability of soil.

k_{20} = Coefficient of permeability at 20°C.

k_T = Coefficient of permeability at test temperature T.

L = Length of sample.

M_f = Fabric Modulus.

n_e = Effective porosity.

O_{max} = Maximum opening size of pipe drain.

p = The average surface contact pressure from wheel load.

$P_{50\ fabric}$ = 50% pore size of filter fabric.

$P_{85\ fabric}$ = 85% pore size of filter fabric.

$P_{95\ fabric}$ = 95% pore size of filter fabric.

q = Rate of flow.

R = Radius, the radius of the circular deflected shape.

r = Inside radius of casing, the radius of hole.

S = Slot width.

s = Shearing resistance.

T = Total time.

T_v = Time factor for given percent consolidation.

t = Elapsed time.

t_f = Fabric tension.

V_d = Discharge velocity.

V_s = Seepage velocity.

W = Estimated rut width.

Z = The thickness of the aggregate layer.

ε_f = Percent strain in fabric.

$\Delta\sigma_{z-f}$ = The differential normal stress across the fabric.

σ_{p-f} = The permissible normal stress on the fabric surface.

σ_z = Actual vertical stress.

σ_{per} = Permissible stress on the subgrade.

μ_T = The viscosity of test water.

μ_{20} = The viscosity of water at $20^{o}C$.

CHAPTER 1

INTRODUCTION

Geotextile has proven effective in a variety of subsurface drainage systems.Geotextiles are ideal filters when compared to conventional filters because permeability characteristics of geotextile readily allow the passage of water and very fine soils while restraining larger soil particles.Geotextile improves the separation of sub,grade soil and drainage aggregate,prevents contamination,controls piping and minimizes the collapse of drain walls.

There are three basic elements of filter criteria for drainage fabrics:

1. Retention Ability
2. Permeability
3. Clogging Resistance.

The clogging is the most serious problem in fabric filter system.There is a relationship between the particle size of the soil and the pore size of the geotextile,thus the clogging may be prevented using proper type of geotextile.

The purpose of this study is to show the relationship between the particle size of the soil and pore size of the geotextile and when the clogging phenomena occurs.

In Chapter 2,general informations about geotextiles,such as types of geotextiles,functions of geotextiles,and design procedure for their separation and support function are outlined.

In Chapter 3,the factors affecting permeability,the methods of determining the coefficient of permeability,and different coefficient of permeability formulas are given.

Chapter 4 includes basic requirements of filters and drains, piping and permeability criteria for filters,and some examples.

In Chapter 5,use of geotextiles as soil filters, general requirements for optimal filter performance, advantages and disadvantages of fabric filters are discussed.

Chapter 6 includes the laboratory work done.In this chapter a description of test apparatus,the characteristics of used soils and geotextiles and test procedure are presented.The results of the performed tests are also given as tables and number of test versus system permeability curves are drawn.

Finally Chapter 7 covers the conclusions derived from this study.

CHAPTER 2

GEOTEXTILES

2.1- GENERAL INFORMATION ABOUT GEOTEXTILES

Geotextiles are synthetic,contemporary construction materials widely used in geotechnical applications;such as drainage,soil reinforcement,support,and erosion control.

There are three types of geotextiles:these are woven,non-woven,and knitted.In a woven geotextile,the fibers or filaments are aligned in two directions,perpendicular to each other.But in a nonwoven,the filaments or fibers are entangled multi-directionally,for this reason nonwoven geotextiles offer the same resistance in all directions.

The major thermoplastic families from which geotextiles are manufactured include the following: Polyolefins (polyethylene, polypropylene) ; Polyesters (polyethylene, terephallate) ; Polyamide (nylon),and Polyaramides. (Thompson,1985).

The prime advantage that geotextiles have over steel and other metals used for geotechnical applications is that polymers do not corrode. However, when this is claimed, the qualyfying statement is often ommited that polymers are nevertheless,susceptible to other forms of attack.

Geotextiles resist mildew, rotting, and insects normally encountered in subsoils. These fabrics are unaffected by soil chemicals, acids and alkalis over a pH range of 3 to 12, and ultra-violet stabilized so giving protection againist deterioration under exposure to natural ultra-violet light. Geotextiles wet or dry, have good tear and puncture resistance and will not shrink, grow or unravel.

When matched to site conditions will reliably perform four main functions:

- Separation and Support.
- Drainage and Filtration.
- Erosion Control.
- Soil Reinforcement.

This categorisation can be confusing in that, in practice, a fabric may perform several functions and its real form of contribution to the solution of the problem is not always clear. For example, in road construction a geotextile may perform simultaneously the roles of separation, reinforcement , and filtration.

2.2- GEOTEXTILES FOR SEPARATION AND SUPPORT

The separation function of geotextile means that hardcore and oversite fill is placed once and stays put. Since geotextile acts as an effective separator, hardcore is not punched into the ground, but retains its original thickness. On areas of the site subject to heavy trafficking problems are often caused by the development of deep ruts.(Gourc,Perrier,and Riondy,1983).

Geotextile improves the load bearing capacity of the subsoil by spreading the wheel loads and acts as a separating medium between the sub-base materials and and the subsoil. Geotextile also prevents the loss of aggregate down into the subsoil and the upward ingress of mud and silt into the road formation.

The poorer the load bearing capacity of the subsoil the greater the thickness of aggregate needed to spread the wheel load over a larger area.The improved support given by the geotextile mat reduces the need for additional aggregate thickness.

It has been known for some time that geotextiles can facilitate the construction of roads there where poor subsoils, heavy rainfalls threatened to slow down the roadworks.

This is especially true for temporary and access roads, which are normally built to a minimum design and remain unpaved. Water will enter the pavement structure and find its way down to the formation soil.Under dynamic wheel loading this can cause soil fines to pump into the sub-base so weakening the pavement.

Due to the separation function of the geotextile , the aggregate does not sink into the subsoil,retains its integrity and bearing capacity,and thus extends the project's life expectancy substantially.

Conversely,field experience has shown that the same design lifetime can be achieved using a thinner layer of aggregate which results in substantial cost savings.(McGown,1983).

Using geotextile reduces overstressing of the formation and therefore minimizes penmanent deflection.Geotextiles are used as separator and support in following constructions:

- Roads,highways,site access roads,farm roads,forest tracks.
- Storage yards,parking lots.
- Airport runways and taxiways.
- Railroads.
- Sports tracks and fields.
- Pipelines,tanks,leeves.

2.2.1 Design Procedure

1. The maximum wheel load and contact pressure anticipated on the surface of the haul road or area are determined.

All design calculations are based on loads applied by pneumatic tires,either a single or dual-tire set,which are assumed to exert a uniform pressure over a circular area of subgrade.The calculations are insensitive enough to aveage contact pressure that a quantity slightly less than the air pressure in the tire can be used as an estimate.

2. The maximum permissible stress on the subgrade is determined.

Initial or localized shear failure of the subgrade will begin when the stress applied to the subgrade reaches times its shear strength.This assumes no confinement of the subgrade soil.Geotextile provides significant soil confinement and allows the aggregate to distribute the load over a somewhat larger area than is predicted by the Boussinesq Theory.

Thus the permissible stress on the subgrade should be adjusted to allow for the effect of the fabric.

$$\sigma_{per} = c(\pi)A \qquad \ldots(2.1)$$

where,

σ_{per} = permissible stress on the subgrade.

c = the shear strength of the soil in psi.

A = coefficient for the soil confining effect of fabric.

The value for A can be adjusted for different acceptable rut depths.Increase the value of A if ruts are allowed to run deep, decrease A must be kept shallow.

3. The rut width and desired maximum rut depth are estimated and from those the geometry of the rut is determined.

The rut width and depth at the surface of the subgrade should be estimated and used to calculate the deflected shape of the fasric.Fortunately,the design calculations for aggregate thickness are somewhat insensitive to rut width,so an estimate based on the track width of tire plus the lateral movement of the tire will suffice.Also since nearly all rutting at the surface of a well compacted aggregate mirrors that occuring in the subgrade, the subgrade rut depth can be estimated as equal to the surface rut depth.

Using these estimates for rut width and depth and assuming a circular arc for the shape of the subgrade rut, the geometry of the rut can be estimated:

$$R = \frac{9W^2}{80d} + \frac{5}{16}d \qquad \ldots(2.2)$$

$$\theta = 2\tan^{-1}\frac{10d}{6W} \qquad \ldots(2.3)$$

where,

R = the calculated radius in inches.

d = the estimated rut depth in inches.

W = the estimated rut width in inches.

θ = the calculated arc expressed in degrees.

4. Using the assumed rut geometry, the percent strain in the fabric is calculated.

By substituting the rut angle and radius into the following equation, the percent strain in the fabric can be calculated.

$$\text{Percent Strain in Fabric, } \varepsilon_f = \left[\frac{4\ R\theta}{135W} - 2\right] 100 \qquad \ldots(2.4)$$

5. The tension in the fabric is determined by multiplying the percent strain by the fabric modulus.

$$t_f = M_f \, \varepsilon_f \qquad \ldots(2.5)$$

where,

t_f = fabric tension in pounds per inch.

M_f = fabric modulus.

ε_f = percent strain in fabric.

6. The differential normal stress carried by the fabric due to the uplifting effect of the fabric under tension is determined.

The differential normal stress between the top side of the fabric and the fabric/subgrade interface summed for the loaded area of the fabric represents that portion of the applied load carried by the fabric. The differential normal stress can be determined from the following relationship:

$$\Delta\sigma_{z-f} = \frac{t_f}{R} \qquad \ldots(2.6)$$

where,

$\Delta\sigma_{z-f}$ = the differential normal stress across the fabric in psi.

t_f = the tension in the fabric in pounds per inch.

R = the radius of the circular deflected shape in inches.

7. The permissible vertical stress on top of the fabric is determined by summing the differential normal stress due to the uplift by the fabric under tension and the maximum vertical stress on the subgrade.

The permissible normal stress on the surface of the fabric is the sum of the permissible stress on the subgrade and the differential stress across the fabric, which can be expressed as:

$$\sigma_{p-f} = \Delta\sigma_{z-f} + A(\pi)c \qquad \ldots(2.7)$$

where,

σ_{p-f} = the permissible normal stress on the fabric surface.

$A(\pi)c$ = the permissible stress on the subgrade.

$\Delta\sigma_{z-f}$ = the differential stress across the fabric.

8. The aggregate thickness required for the imposed wheel load is determined using the Boussinesq Theory.

$$\sigma_z = p\left[1-\left(\frac{1}{1+\left(\frac{a}{Z}\right)^2}\right)\right]^{3/2} \qquad \ldots(2.8)$$

where,

p = the average surface contact pressure from wheel load in psi

Z = the thickness of the aggregate layer in inches.

a = the radius of the loaded area determined from the following equation:

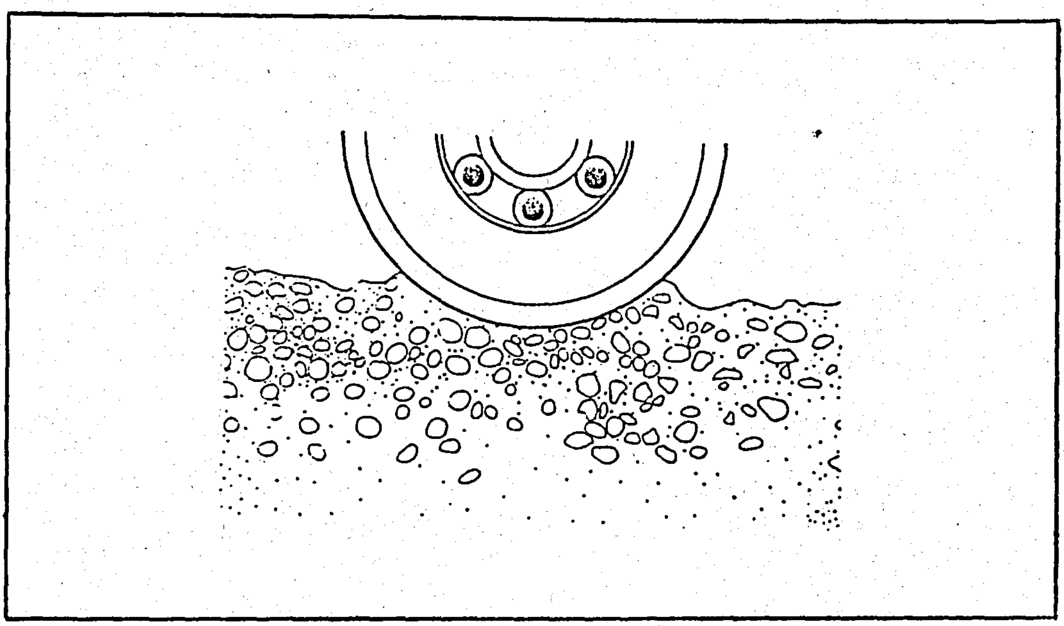


FIG 2.1 HEAVY WHEEL LOADS COMPRESS BASE COURSE INTO SUBSOIL

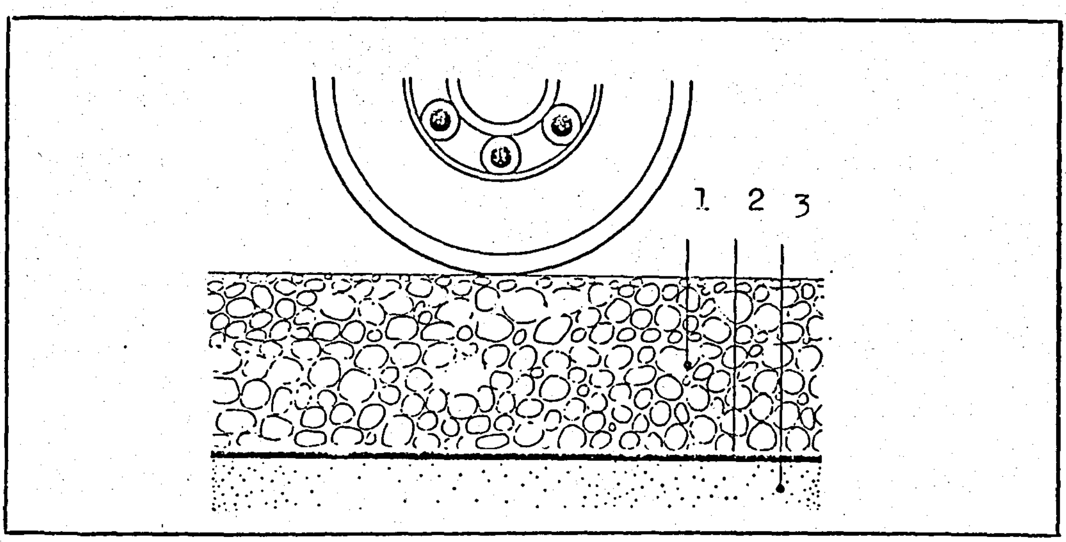


FIG. 2.2 GEOTEXTILE SEPARATES AND SUPPORTS

1. Compacted Aggregate
2. Geotextile
3. Subsoil

2.3- GEOTEXTILES FOR DRAINAGE AND FILTRATION

Geotextile has proven effective in a variety of subsurface drainage systems. It is being used world-wide in French, blanket, foundation, and toe drains for highways, airfields, railroads, mines, dams, leeves, parking lots, buildings, and storage yards.

Geotextiles are ideal filters when compared to conventional filters because permeability characteristics of geotextile readily allow the passage of water and very fine soils while restraining larger soil particles. Geotextile improves the separation of sub-grade soil and drainage aggregate, prevents contamination, controls piping and minimizes the collapse of drain walls. It has a more consistent uniformity and is more permeable than graded filters aggregates, which often vary from quarry to quarry. Geotextile is also more economical than conventional filters because it eliminates the need for costly sands, well graded aggregates, and perforated pipe in certain situations. It permits faster and simpler construction methods by eliminating the need for slant wall ditches and shoring. Geotextile is more readily available and more suitable to a wider range of soil conditions than conventional filters.

Fabric filters eliminate the need for aggregate filters. Both single-component and multiple-component aggregate filters can be replaced by a single fabric filter layer. The fiber structure of fabric filters provides the same particle retention

Graded aggregate systems (usually multilayred) are used with limited success beneath armor as both energy dissipators and filters.Such aggregates are generally difficult to source and install,resulting in relatively high in-place costs.Even when properly installed,graded aggregates are susceptible to erosive forces.As the ability of both the armored system and the protected structure are endangered by the loss of any aggregate layer.

Erosion control fabrics are reliable and economical alternative to aggregate systems.Fabrics act as energy dissipators by shielding the slope from the erosive forces of moving water.Furthermore,as filters,the fabrics allow adequate groundwater seepage to pass from the protected slopes while retaining underlying soil particles.Fabrics will not wash out beneath the armor;thus,they form reliable filtration and energy dissipation systems.

2.4.1 Design Considerations

Sufficient fabric strength is required to resist damage during armor installation and subsequent service life. In addition,where the application mandates that the fabrics act as a filter,selection of appropriate fabrics should be based on established filter criteria.

1. Fabric Strength Requirements

Installation stresses imposed on the fabric vary significantly with armor size,angularity,and placement procedures.An erosion control fabric may be subjected to ignificant stresses and abrasive forces during its service life. evere storms often cause armor rocking on coastal and large nland lake protection systems. Armor movement is synonymous ith high fabric stresses and abrasive forces. Severe abrasive orces can also develop in scour protection applications due o the movements of suspended or bottom-rolling particles.

2. Fabric Filtration Requirements

Where application mandates filtration by the armored rotection system, selection of the fabric should be based on ts adherence to established filter criteria as well as strength equirements. These are three basic elements of filter criteria or fabrics:

. Retention Ability (Piping Resistance).

$$\frac{EOS_{fabric}}{D_{85\ soil}} < 2 \text{ or } 3 \qquad \ldots(2.11)$$

here,

EOS_{fabric} = Equivalent Opening Size of fabric

$D_{85\ soil}$ = 85% size of protected soil.

. Permeability.

$$k_{fabric} \gg 10k_{soil} \qquad \ldots(2.12)$$

here,

k_{fabric} = coefficient of permeability of fabric.

k_{soil} = coefficient of permeability of protected soil.

.Clogging Resistance.

$GR \leqslant 3$ ...(2.13)

where,

GR = Gradient Ratio.

3.Other Design Considerations

In addition to strength requirements and filter criteria, other factors influence the performance of armored erosion protection systems:

.Bedding Materials

Properly sized and placed bedding materials e.g.,crushed stone,sand or locally available gravel will significantly reduce both installation and in-service fabric stresses.Bedding material is an integral part of the armored system's hydraulic functioning.Without bedding,large armor can cause the fabric to bridge over gullies and depressions on the slope.Water can then flow behind the fabric,possibly causing erosion. The uniform pressure exerted by a bedding material,relative to the armor,provides a better fabric-soil interface conformity, thus reducing potential erosion or uplift pressures caused by trapped pockets of water.Relatively small-sized armor will exert a more uniform pressure on the fabric and,thus provide good fabric soil conformity:bedding materials may not be required for such applications.

When bedding material is used,it should be sized to resist the erosive forces remaining after water passes through the armor.If improperly sized,the bedding material may wash out from beneath armor and subject the fabric to significantly greater stresses.A loss of bedding material can also reduce the potential for good fabric-soil conformity.

.Fabric Toe-ins

Fabrics should be securely toed-in at the top of the slope.Similarly,the fabrics should be securely toed-in Or toe-wrapped at the slope bottom.Proper toe-ins will significantly reduce the potential for water flowing freely between the fabric and protected soil slope,thus reducing the potential for erosion.The top toe-in should be placed at an elevation above which the dynamic water forces from the design wave (current) and high water level will be comparely diminished. The bottom toe-in (toe wrap) should be placed below the elevation at which the design wave(current) and low water level will cause scouring.Variations from these elevations may be warranted by the application of the armored protection system or by the local hydraulic conditions.

.Restricted Flow Area Through Fabric

The area of the fabric through which water can flow from the protected slope may be critical.Specifically,precast, preassembled concrete armor can restrict flow in as much as 85% of the fabric surface,leaveing only a small percentage of the area available for drainage.The design engineer should be rewiev this condition.His design should specify a having sufficient water flow capabilities through the reduced area or a bedding material should be placed beneath such armor to allow for drainage through a significantly greater fabric area.

The use of geotextiles in some protective engineering works,such as,offshore breakwater,Stone revetment, Gabion revetment,and Retaining Walls,are shown through the Figures 2.3.,2.4.,2.5., and 2.6.

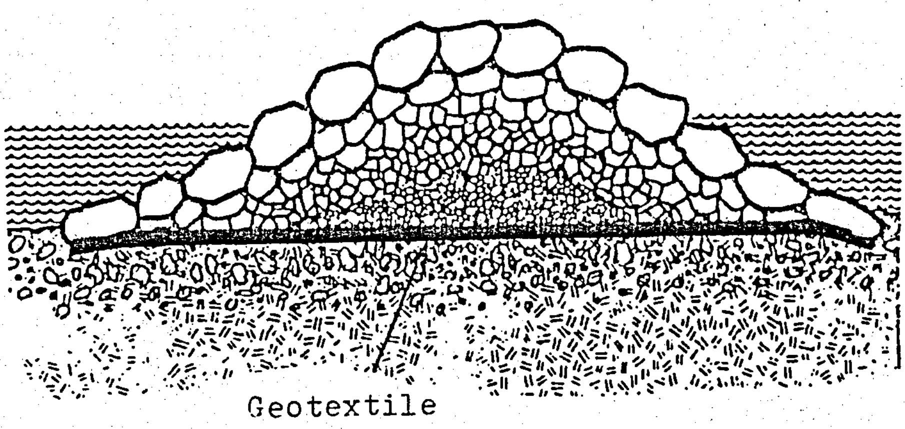


FIG. 2.3. THE USE OF GEOTEXTILE IN OFFSHORE BREAKWATER

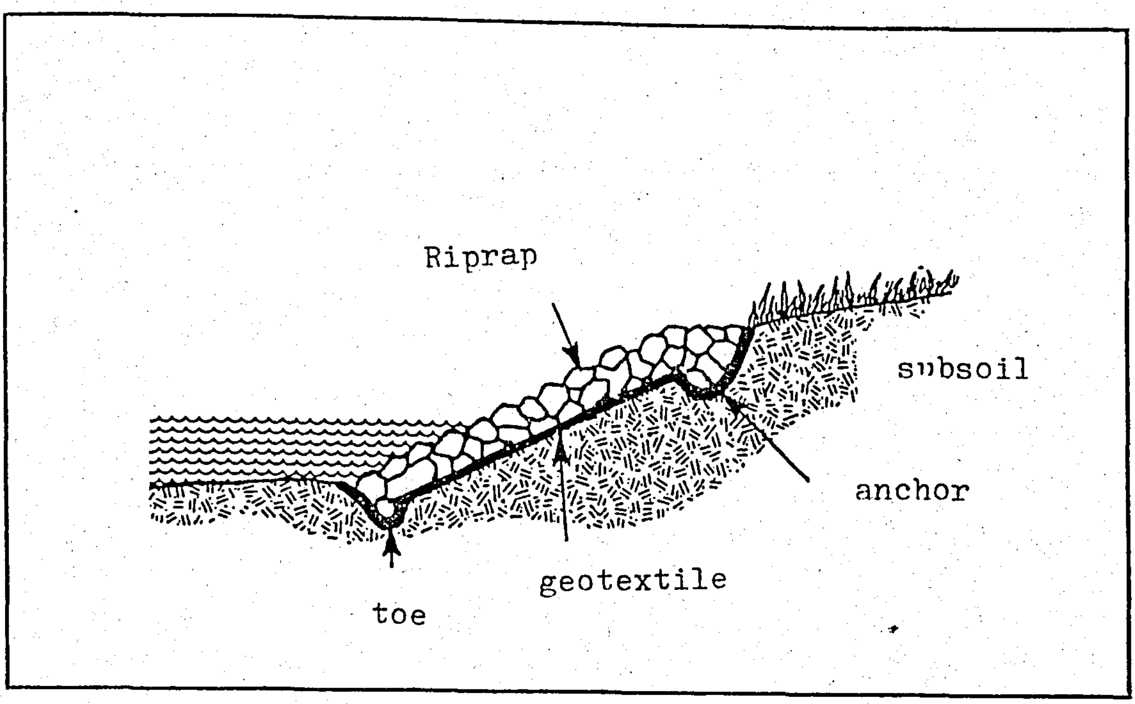


FIG 2.4 USE OF GEOTEXTILE IN STONE REVETMENT

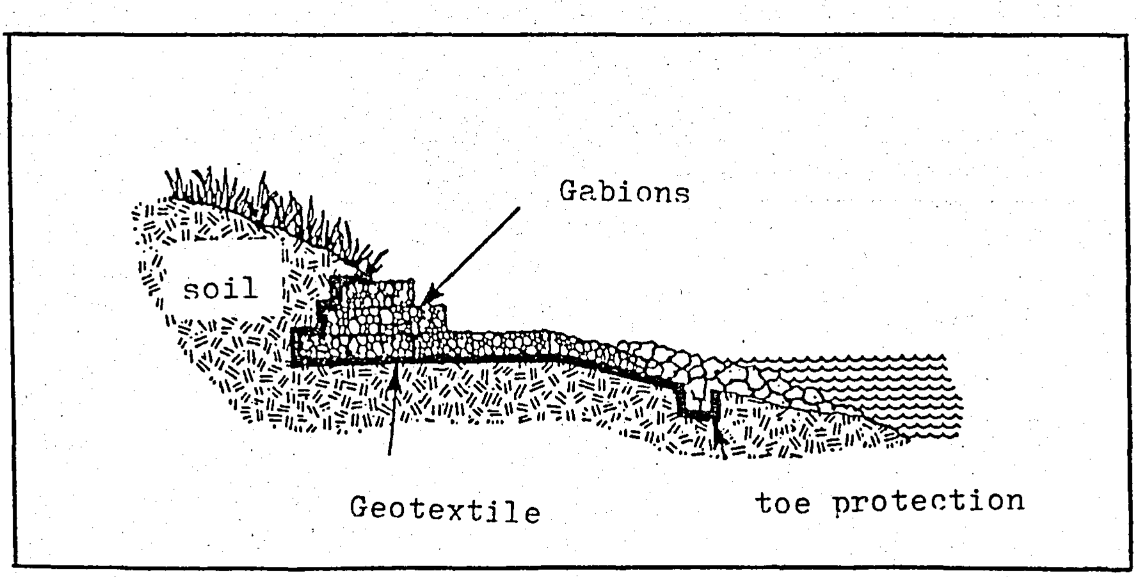


FIG 2.5 USE OF GEOTEXTILE IN GABION REVETMENT

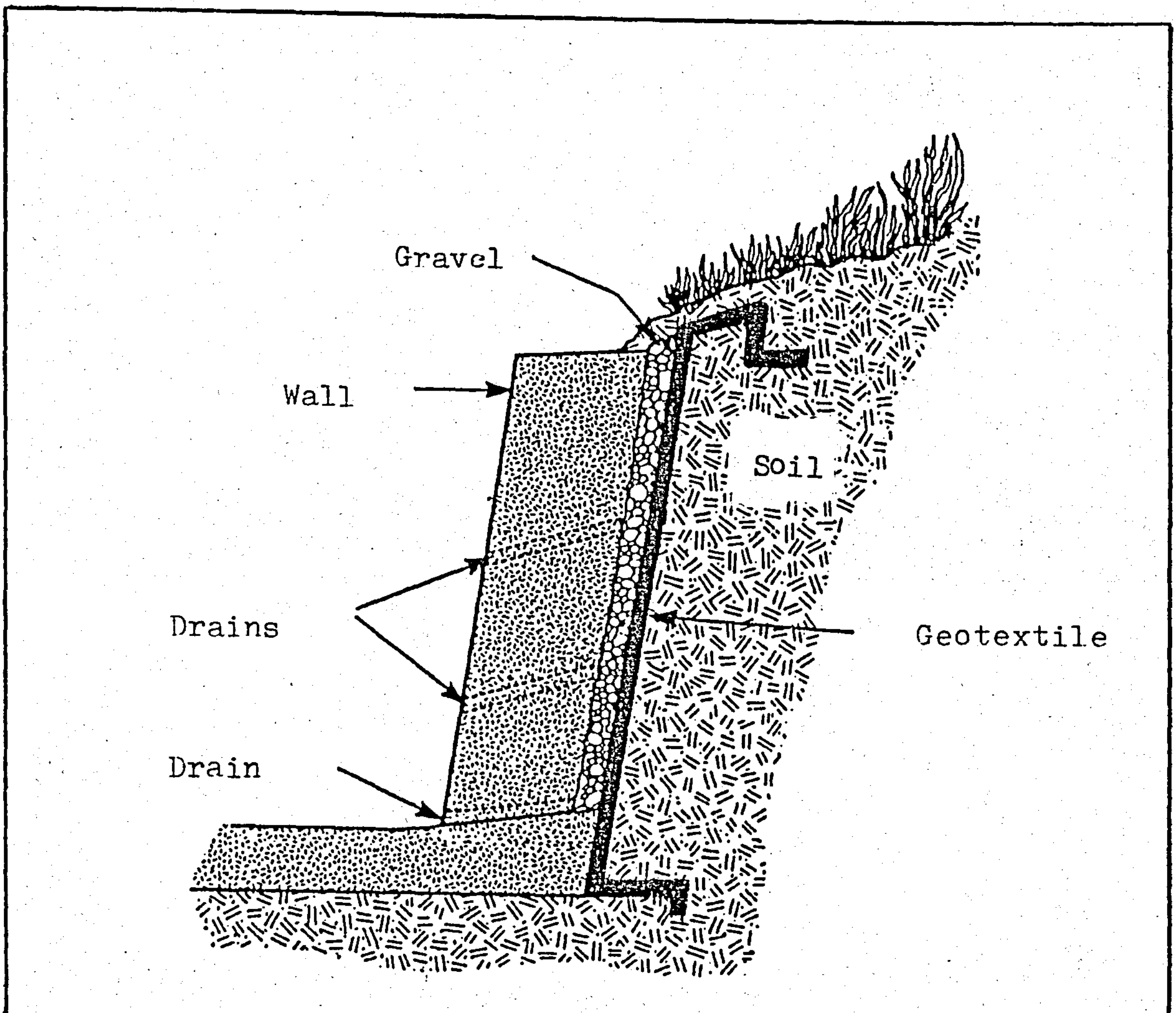


FIG 2.6 USE OF GEOTEXTILE IN RETAINING WALL

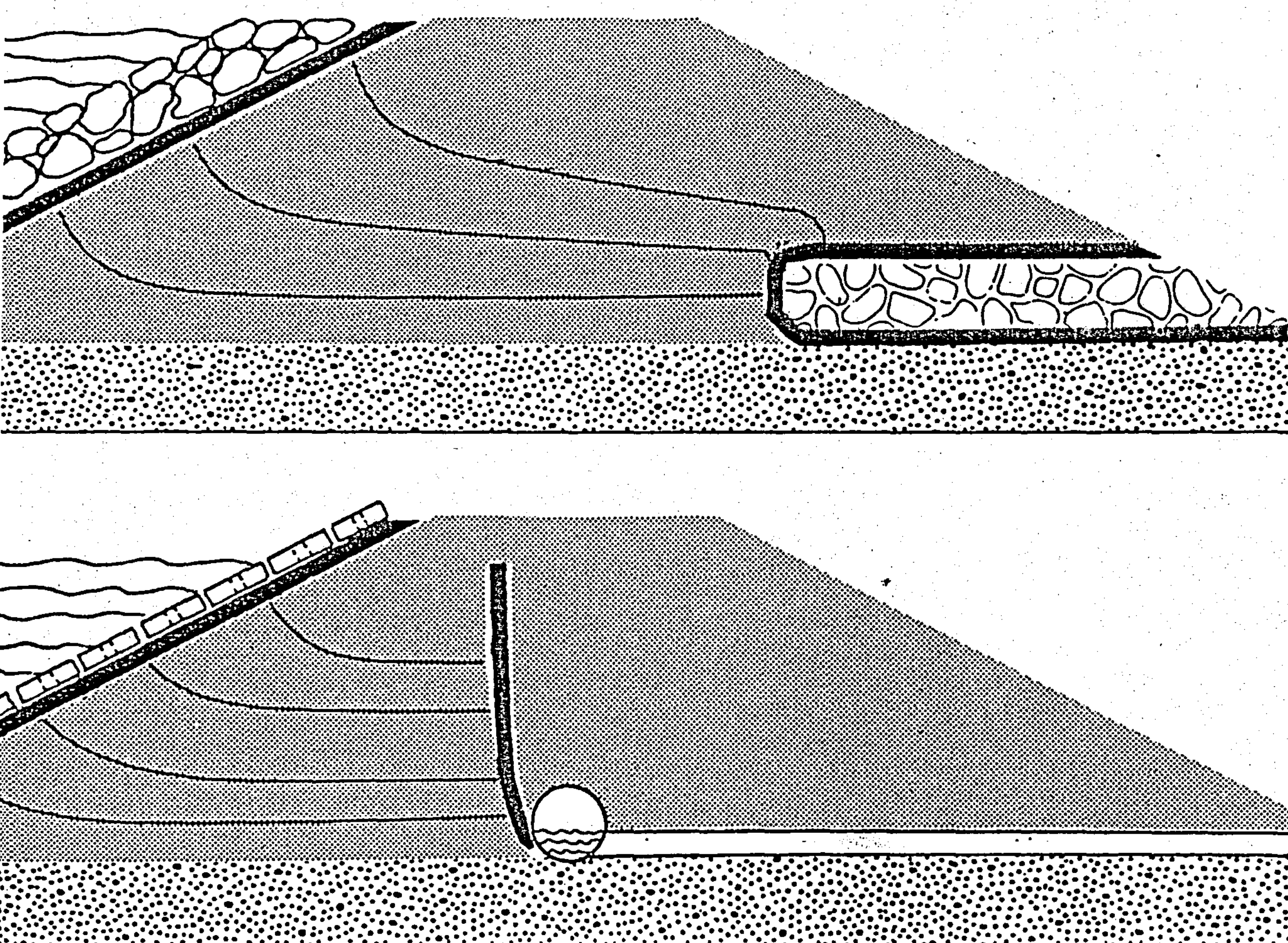


FIG 2.7 PROTECTION OF THE UPSTREAM FACES OF EARTH DAMS

2.5- GEOTEXTILES FOR SOIL REINFORCEMENT

Embankments on soft foundations often require some type of reinforcement for stability.Reinforcing embankments with fabrics has been found to increase their stability,especially for low embankments with high live loads.Differention between the fabric functions of separation and reinforcement is difficult.Generally,woven geotextiles or geogrids are used for soil reinforcement because of their rough surfaces(Quast,1983).

The primary influence of the geotextile reinforcement is to reduce both the shear stresses in the soft foundation and the vertical differential settlements of the top of the embankment.Total settlements are only slightly affected by the reinforcement.The degree of improvement due to the presence of the reinforcement is more pronounced with higher modulus geotextiles.

2.5.1 Reinforcement Orientation and Location

Application of the simple Mohr Circle of strain to the analysis of normal strains on sets of horizontal and vertical planes within an unreinforced embankment fill show that for quite modest values of shear strain the minor principal strain will be tensile.Furthermore,such tensile strains will not be horizontal particularly in zones close to the face of the embankment slope and in vicinity of the toe.Since the objective is to install geotextiles to act as an effective tensile reinforcement they must be located within the tensile zones and orientated in the directions of principal tensile strains. Assuming stress and strain axes to be coincident published stresses and strain distributions,may be used to detrmine the appropriate reinforcements directions.Horizontal layers of reinforcement would be correctly aligned within the main body of any embankment they would have inappropriate inclinations under the batter especially near the toe.It has been pointed out that reinforcing outside the tensile zones can encourage, rather than prevent failure.Although this may be so this notion only pertains to reinforcement placed in a discrete horizontal layers which leave the face of the batter exposed.If alternatively the reinforcement is temporarily extended beyond the face of the batter there is the possibility of completely encapsulating the batter face and anchoring the free end of the geotextile in the soil layer above by a suitble bond length.

The majority of geotextiles are manufactured using the polyolefins, or polyester.All are prone to loss of strength on exposure to ultra-violet light,although this potential problem can be reduced by various manufacturing techniques.Both polymers offer generally excellent resistance to environmental attack. However,each has its own particular weakness.The polyolefins, especially polypropylene, tend to display undesirable creep properties. Unless a sufficiently high factor of safety is applied this could lead to creep rupture within the required design life. Polyester exhibits better creep characteristics but in certain forms is subjected to loss of strength when allowed to absorb water. In the design process allowance must be made for loads induced at the construction and in-service phases. Both these stages will involve settlement which will induce strain in the reinforcement. Further strains would be caused by trafficking and dumping of fill. These are likely to be much larger and more localised than those due to settlement. Finally compaction of reinforced fill car cause large geotextile strains. This has been confirmed in the field where compaction induced strains up to 6% have been observed.

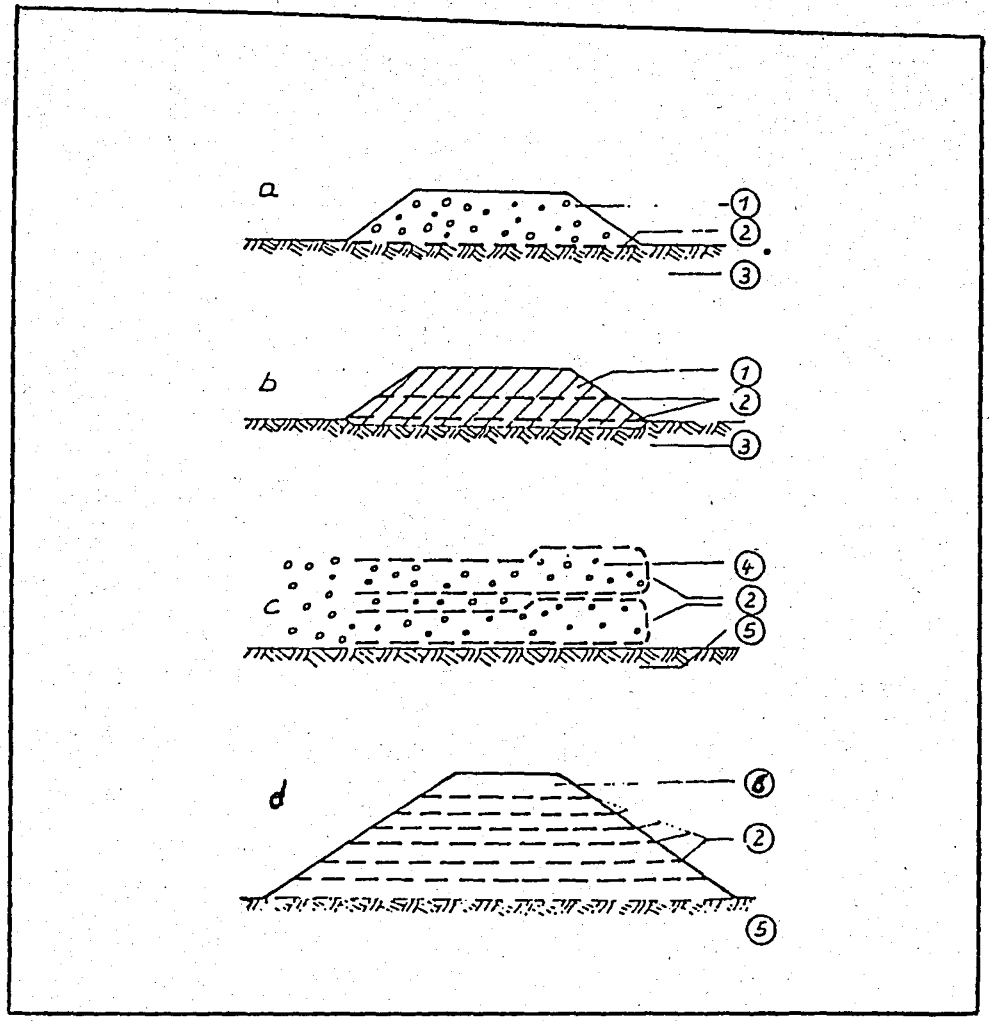


FIG.2.8 APPLICATION POSSIBILITIES FOR GEOTEXTILES IMPROVING FOUNDATIONS AND EARTHWORK CONDITIONS

a-Interface

b-interface and reinforcing element

c-Reinforcing element

d-Drain and reinforcement element

1-Embankment

2-Geotextile

3-Foundation of low load carrying capacity

4-Soil

5-Foundation

6-Soil with high water content and high plasticity

CHAPTER 3

PERMEABILITY

3.1- GENERAL CONSIDERATIONS

A permeable material is one which is capable of being penetrated or permeated by another substance,which usually is a gas or liquid.Thus,dry cement is permeable to air permeability test is a useful means of obtaining an indirect measure of its fineness of grind ,since the speed of flow of air through it can be related to the size of the pore spaces between the particles. Likewise,soils and aggregates,and jointed,or vesicular rocks often are permeable to air and water.Many materials allow themove-movement of fluids by a diffusion process,but that is not within the meaning of permeability as used in soil mechanics.In the study of soil mechanics,a material is considered permeable if it contains interconnected pores,cracks,or other passageways through which water or gas can flow.A rock may be virtually impervious,yet contain cracks or joints which make a formation highly permeable to the flow of water.In fact,the permeability of most rock abutments and dam foundations is determined almost entirely by the joint and crack patterns.And many clays are extremely resistant to the flow of water,yet shrinkage cracks or interbeds of silt or sand may increase their permeabilities thousands of times.

The permeability of a soil is one of its most fundamental and important properties.It enters into nearly all seepage,settlement,and stability problems confronting the soil engineer.The amount of leakage through and under dams,the rate at which the strength of a deposit increases after it has been subjected to a consolidating pressure are typical of the many problems in which the permeability of a soil can be a critical factor. (Cedergren,1977)

The importance of evaluating the permeability of a pervious soil has been long recognized and test techniques for measuring it have been well developed and are widely used.Soils with permeabilities of less than 1 micron per second are often considered "impervious." More use is being made of"impervious" soil to line canals and reservoirs and to construct cores for earth dams.

Many of the design and construction problems associated with hydraulic structures and engineering works involving drainage are caused by imbalances of permeabilities of earth and rock masses.Frequently water can enter spaces behind walls,under pavements and canal linings more readily than it can escape,thus creating conditions detrimental to safety and performance.Water that becomes trapped in earth and rock masses contributes to landslides,and is a serious threat to stability during earthquakes. Undetected joints or strata of high permeability in the foundations or abutments of dams create serious leakage and uplift problems. (Scott,1963).

3.2- COEFFICIENT OF PERMEABILITY

Coefficient of Permeability,or DARCY's Coefficient, is defined as the discharge velocity through unit area under with hydraulic gradient.It is a term in Darcy's Law for laminar flow in porous media,

$$Q = kiAt \qquad ...(3.1)$$

In equation 3.1 , Q is the quantity of seepage in a cross section having an area of A normal to the direction of flow,under a hydraulic gradient i,during a length of time t.The coefficient of permeability k is equal to the discharge velocity under a hydraulic gradient 100%.By arranging the terms,Eq.3.1 provides the basis for many experimental determinations of permeability that measure seepage quantity:

$$k = \frac{Q}{iAt} \qquad ...(3.2)$$

Darcy's discharge velocity multiplied by the entire cross sectional area including voids e and solids 1 gives the seepage quantity Q under a given hydraulic gradient $i = \Delta h/\Delta l$ or h/l.It is an imaginary velocity that does not exist anywhere. The average seepage velocity V_s of a mass of water progressing through the pore spaces of a soil is equal to the discharge velocity ($V_d = ki$) multiplied by (1 + e) / e , or the discharge velocity divided by the effective porosity n_e ; hence the permeability is related to seepage velocity by the expression:

$$k = \frac{V_s n_e}{i} \qquad ...(3.3)$$

For any seepage condition in the laboratory or in the field where the seepage quantity,the area perpendicular to the direction of flow, and the hydraulic gradient are known, the coefficient of permeability can be calculated.Likewise, for any condition where the seepage velocity is known at a point where the hydraulic gradient and soil porosity also are known,permeability can be calculated.

Experimentally determined coefficients of permeability can be combined with prescribed hydraulic gradients and discharge areas in solving practical problems involving seepage quantities and velocities.When a coefficient of permeability has been properly determined,it furnishes a very important factor in the analysis of seepage and in the design of drainage features for engineering works,

The engineers' coefficient,which is used in practical problems of seepage through masses of earth and ather porous media applies only to the flow of water and is a simplification that is introduced purely from the standpoint of convenince.It has units of a velocity and is expressed in centimeters per second, feet per minute,feet per day,depending on the habits and personal preferences of individuals using the coefficient.In standard soil mechanics terminology k is expressed in centimeters per second.

A clayey soil with very fine grains will have a very much lower permeability coefficient than will a sand with relatively coarse grains,even though the void ratio and the density of two soils may be the same.The reason is the greater resistance offered by the very much smaller pores or flow channels in the fine-grained soil through which the water must pass it flows under the influence of a hydraulic gradient.From this standpoint,it may be said that the coefficient of permeability is independent of the void ratio or density when we are comparing soils of different textural characteristics.On the other hand,when we consider the same soil in two different states of density,the permeability is dependent on the void ratio,since the soil grains are brought into closer contact by the process of compaction and densification.The pore spaces are reduced in size,and resistance to flow is increased.

Attention is directed to the fact that,in the application of the DARCY Law,the cross-sectional area A is the area of the soil including both solids and void spaces.Obviously,the water cannot flow through the solids,but must pass only through the void spaces.Therefore,the velocity ki is a fictitious velocity at which the water would have to flow through the whole area A in order to yield the quantity of water Q which actually passes through the soil. Thisfictitious velocity is referred to as "the velocity of approach" or the "superficial velocity" of the water just before entering,or after leaving the soil mass.

A dimensional analysis of the Darcy's Law indicates that the coefficient of permeability k has the dimensions of a velocity,that is,a distance divided by time.Therefore,permeability is sometimes defined as "the superficial velocity of water flowing through soil under unit hydraulic gradient".(Spangler,1966). The coefficient of permeability often is considered to be a constant for a given soil or rock,it can vary widely for a given material,depending on a number of factors.Its absolute valuess depend,first of all,on the properties of water ,of which viscosity is the most important.For individual materials and formations,its value depends primarily on the dimensions of the finest pore spaces through which water must travel and on the size of cracks , continuity of cracks,and joints in rocks, fissured clays,etc.In short the ease with which water can travel through soils and rocks depends largely on:

1.The viscosity of the flowing fluid(water).

2.The size and continuity of pore spaces or joints through which the fluid flows,which depends in soils upon:

a.The size and shape of the soil particles.

b.The detailed arrangement of the individual soil grains, called the structure.

3.The presence of discontinuities.

3.3- FACTORS INFLUENCING PERMEABILITY

When water flows through a pipe or open channel,the velocities near the edges are considerably smaller than those in the center of the flowing stream,but when water flows through homogenous soils or other porous media under uniform gradients the average velocities are no greater at the center of a formation than at its edges.Flow in pipes and conduits is almost always non-turbulent whereas in soils and aggregates it is always non-turbulent or laminar.

Whenever a fluid is in motion,layers of the fluid slip and move in relation to other layers.The ease with which they slip depends on the viscosity of the fluid,which is the resistance or "drag" offered to motion.The viscosity of water,like that of most fluids,is reduced at high temperatures,the range is much narrower than for other fluids(See Fig.3.1).It is customary to standardize permeability values at 20^{o}C or 70^{o}F and make a correction if field temperatures are substantially different.

By application of laws of physics it can be shown that the resistance to viscous or streamline flow increases in direct proportion to viscosity, and the velocities attained by seeping fluids vary inversely with viscosity. When flow is viscous or laminar, layers and shearing resistance is related to viscosity according to Newton's Law of Friction:

$$s = u\frac{dV}{dr} \qquad \ldots(3.4)$$

where;

s = shearing resistance

u = the absolute viscosity in poises.

$\frac{dV}{dr}$ = the change in velocity in a distance dr normal to the direction of flow.

The actual pore channels through which water finds its way through soils are very tortuous and often semidiscontinuous and the hydraulics of flow through such channels is extremely involved. By making simplifying assumptions efforts have been made to calculate the permeabilities of simulated soils purely from theoretical considerations. The main value of such efforts has been in disclosing fundamental relationships that govern flow through minute pore spaces.

Darcy (1856) investigated this problem experimentally by using a simple apparatus to force water through small specimens of sand (See Fig. 3.2). Darcy's experiments demonstrated that the rate of flow q through the sand varies in direct proportion to the cross-sectional area A of the specimens and to the difference between the hydrostatic head at two ends of specimens and is inversely proportional to the length of the column of sand tested.

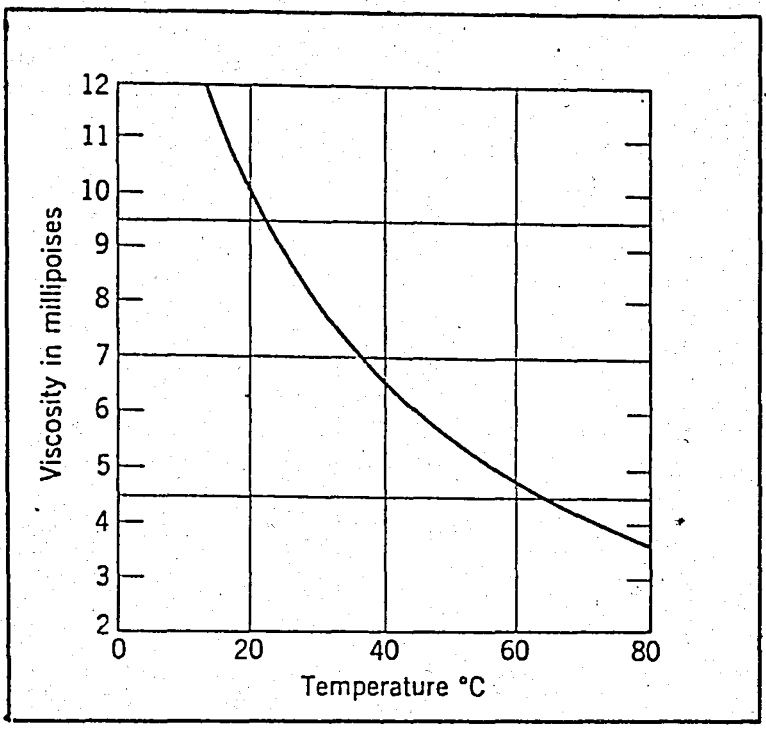


FIG.3.1 VISCOSITY OF WATER (Cedergren,1977)

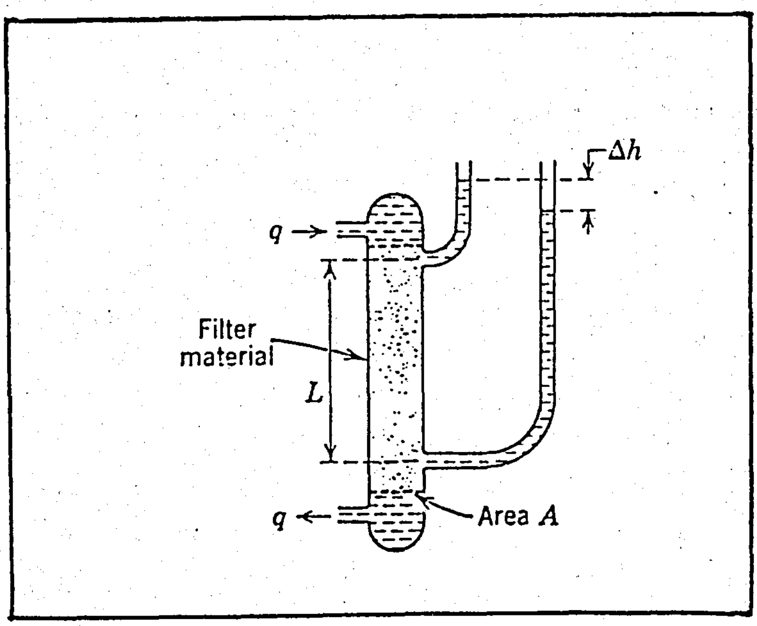


FIG.3.2 DARCY'S TEST APPARATUS (Cedergren,1977)

These relationships can be expressed as,

$$q \sim \frac{A\Delta h}{L}$$

or

$$q=(\text{a constant})\,\frac{A\Delta h}{L}$$

Darcy's experiments produced this simple relationship which has since become known as Darcy's Law.One of the common forms of Darcy's Law is

$$Q= kiAt \qquad ...(3.1)$$

3.3.1 Influence of Grain Sizes

From theoretical considerations it has been shown that permeability can be expected to vary with the squares of the diameters of pore spaces and the squares of the diameters of soil particles.The permeability of soils varies significantly with grain size and is extremely sensitive to the quantity, character,distribution of the finest fractions.

3.3.2 Influence of Particle Arrangement(Structure)

The arrangement of soil particles can influence permeability in two ways:

1. By sorting or stratification
2. By detailed orientation of particles and the balling up of fines or broad dispersion of the fines.

Natural soil deposits are always more or less stratified or non-uniform in structure.Water deposited soils are usually constructed in a series of horizontal layers that vary in grain size distribution and permeability.

3.3.3 Effect of Openwork Gravel

An indication of the presence of openwork gravel in a soil formation is given by the behavior of the water table.The extremely high permeabilities of these formations,which contain no fines,often permit rapid equalization of hydrostatic pressures ,thus allowing the water table to rise and fall almost as quickly as an adjacent system.

3.3.4 Influerce of Dispersion of Fines

The detailed arrangement of soil particles can have a major influence on permeability and other soil properties;for example,if soils are compacted in a relatively dry state, a comparatively harsh permeable structure is usually formed.On the other hand,if liberal amount of moisture are present,the particles tend to slide over one another into a relatively well -knit,smooth,impermeable type of structure.

3.3.5 Influence of Density

Density,also void ratio or porosity,of soil masses,though less important than grain size and soil structure,can have a substantial influence on permeability.The denser a soil and the smaller the pores,the lower its permeability.Often in the construction of reservoirs the soil at the bottom of the reservoir is compacted in place to improve watertightness.In the construction of dams and leeves through compaction of fill materials is required to obtain strong embankments and to ensure the best possible watertightness of impervious zones.As a rule, the narrower the range of particle sizes,the less permeability is influenced by density(See Fig.3.3).

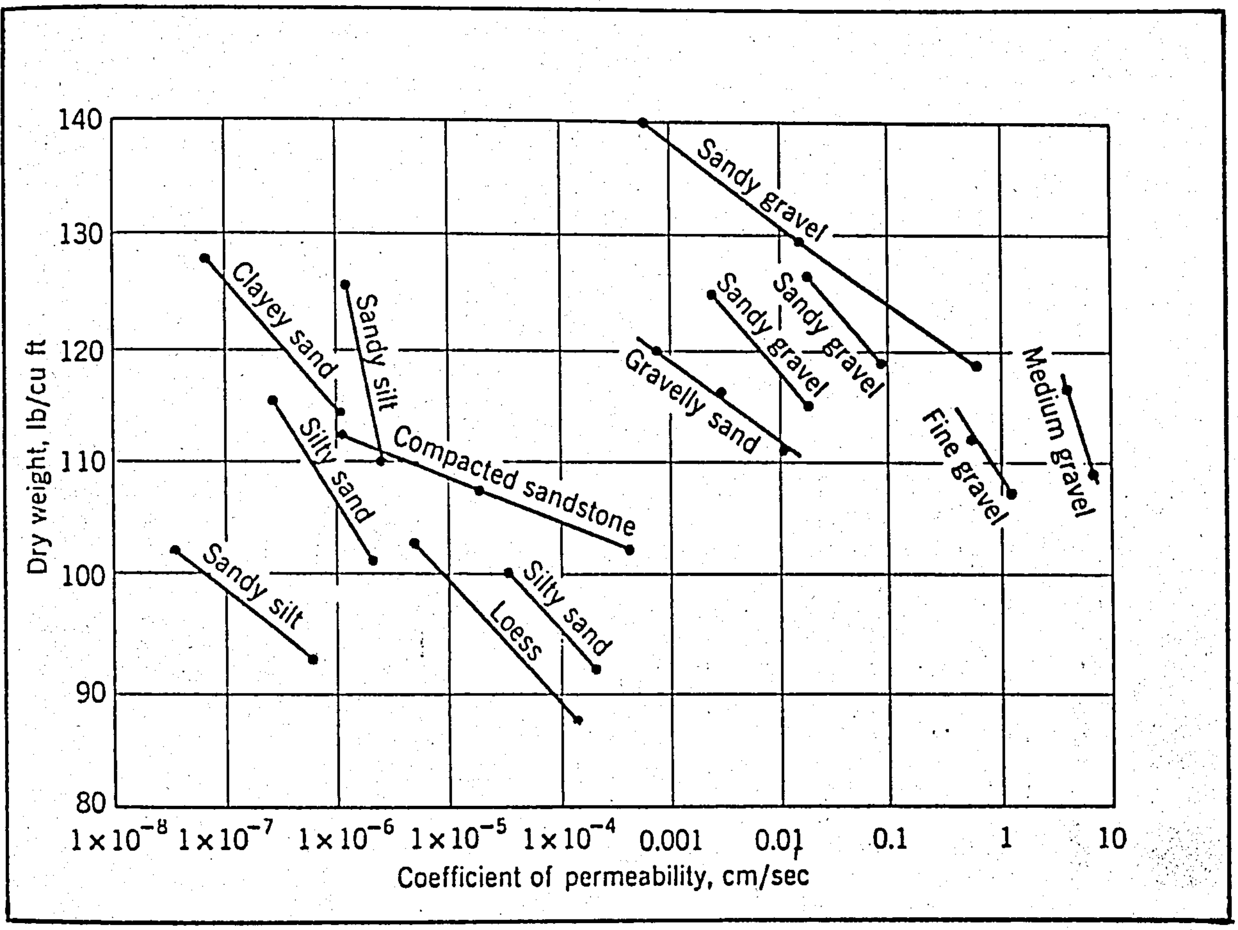


FIG.3.3 RELATION BETWEEN COEFFICIENT OF PERMEABILITY AND SOIL TYPE AND DENSITY(log scale)

BLends of sand and gravel often used for drains are virtually useless as drainage aggregates if they contain more than insignificant amounts of fines.Sometimes in constructions that require the use of permeable,free-draining aggregates or embankment materials attempts are made to utilize materials of borderline permeabilities by keeping compaction low.If high permeability is needed in a drain or embankment material,it is a mistake to obtain it at the expense of good compaction,for poorly compacted materials have low strength and high compressibility. Serious sloughing can occur when they become saturated,and they are highly vulnerable to liquefaction during earthquakes,

While the consolidation process is going on foundations their permeabilities are becoming less.Generally,decreases in the permeabilities of clay foundations are rather moderate,but they can be large in highly compressible organic silts and clays and in peats.

3.3.6 Influence of Discontinuities

Compact clays are often contain shrinkage or shear cracks that tender such formations thousands of times more permeable than the clay between the cracks.Likewise,jointed rocks often have mass permeabilities many times greater than the basic materials between joints.Major dam failures have been caused by some unknown seam or joint system that fed water under pressure into abutments or allowed piping to occur.If seepage quantities increase with the passage of time at a given head, steps should be taken to reduce the permeability by grouting, constructing impervious blankets,or other suitable methods.

3.3.7 Influence of Size of Soil or Rock Mass

When the coefficients of permeability of earth masses are being determined in the development of projects in which seepage conditions will be changed by the project , it is important that the scope of the study be adopted to the size of the soil or rock masses that will influence seepage behavior. Therefore,it is important that all formations influencing the seepage be investigated.If only small specimens are obtained from test holes,the answers will be representative of the overall mass only if the samples are representative of the mass.

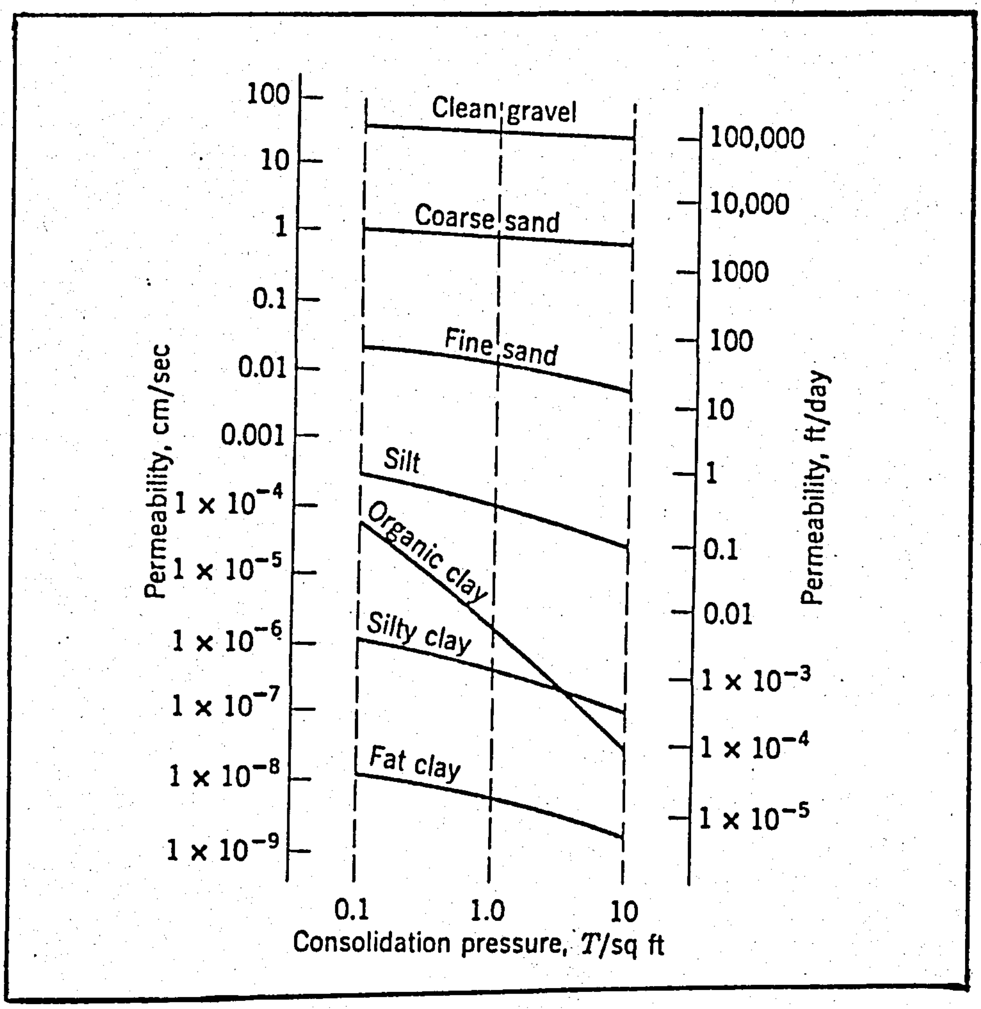


FIG 3.4 PERMEABILITY VERSUS CONSOLIDATION PRESSURE

3.4 INDIRECT METHODS FOR DETERMINING PERMEABILITY

The permeabilities of clays and silts can be calculated from data recorded in laboratory consolidation tests by using the following relationship developed by Terzaghi:

$$T_v = \frac{k}{\gamma_w m_w} \frac{t}{H^2} \qquad \ldots(3.5)$$

where;

T_v = time factor for agiven percent consolidation.

k = the coefficient of permeability.

γ_w = the density of water.

t = the kime required to reach the given percentage of consolidation.

H = the longest drainage path.

The permeability of clean filter sand can be calculated from a number of formulas such as the following developed by Hazen (1911):

$$k = C_1 \, D_{10}^2 \qquad \ldots(3.6)$$

where;

k = the coefficient of permeability in cm / sec.

C_1 = a factor varying from about 90 to 120 (often 100)

Frequently the permeability of clays and silts is determined directly by using the consolidation test apparatus as a falling head permeameter. (see Fig.3.4 on page)7

3.5- LABORATORY METHODS FOR DETERMINING PERMEABILITY

The coefficient of permeability of soil and rock masses can be determened by any controlled test in which the cross sectional area, the hydraulic gradient, and the quantity of flow are known or can be approximated. The permeability can be computed from Darcy's Law,

$$k = \frac{Q}{iA} = \frac{q}{i\,A}$$

Laboratory permeability tests used commonly are the constant head and falling head types.

3.5.1 Constant Head Permeability Test

The constant head permeability test is more applicable to permeable materials such as filter or drain aggregates. A specimen of the material is placed in a cylindirical mold and a continuous supply of water is fed through the sample. The water that passes through the sample in time t, flows in a container, where it is collected and the rate q, calculated. Performing this test, the coefficient of permeability of soil can be determined from following equation:

$$k = Q\frac{L}{hAt} \qquad \ldots(3.7)$$

where,

q = the rate of flow

L = the length of sample

h = the net hydrostatic head

A = the area of the cylinder

In computing the value of the permeability coefficient from data obtained in a test of this type,as in all permeability problems,it is important to keep the computations dimensionally correct.A relatively easy and sure way to do this is to decide in advance the units in which the coefficient of permeability is desired.Then reduce the values of Q and A to those units before making the computation.

3.5.2 Falling Head Permeability Test

If the permeability is low,the time becomes excessive and evaporation during the test introduces errors in the results. Low-permeability soils can be tested in the laboratory by the falling head test.

A specimen is placed in a tubular chamber of suitable diameter and connected with a suitable overflow arrangement and collection container.A small diameter standpipe tube is connected to the top of the larger tube;its diameter is adjusted to the permeability of the material being tested.

In making a test with a falling head type of apparatus, the standpipe is filled to a level somewhat above point P.When it is at P,a stopwatch is started and the time required for the water level to drop to one or more lower points is recorded.

As a result of this test,the calculation of permeability can be set up as follows:

$$dQ = -a.dh = k\frac{h}{L} A\ dt$$

By transposing terms,

$$k = \frac{a.L}{A\ dt} \ln \frac{h_1}{h_2} \qquad ...(3.8)$$

This is the general equation for computation of permeability from a falling head test.

$$k = 2.3 \frac{aL}{Adt} \log \frac{h_1}{h_2} \qquad ...(3.9)$$

where,

k = coefficient of permeability of tested soil at the temperature T in cm / sec.

a = area of the used standpipe in cm^2.

L = height of the sample in cm.

A = cross-sectional area of the sample in cm^2.

h_1 = initial hydrostatic head in cm.

h_2 = final hydrostatic head in cm.

dt = elapsed time in sec.

Frequently, as permeability tests are run, the measured permeability becomes progressively smaller. When this is the case air from the test water is probably filling the voids in soil, causing air locking. A considerable amount of air is usually present in ordinary tap water. The use of distilled water at higher than room temperature eliminates air locking.

When testing soils for permeability in the laboratory, it is necessary to hold the number of variables to a minimum. One minor variable, the viscosity of water, is standardized by performing tests at $20^{o}C$ or by making a correction for tests performed at other temperatures. The correction is as follows:

$$\frac{k_{20}}{k_T} = \frac{\mu_T}{\mu_{20}}$$

or rearranging,

$$k_{20} = k_T \frac{\mu_T}{\mu_{20}} \qquad \text{...(3.10)}$$

where,

k_{20} = coefficient of permeability at 20^{o}C(standard temp.)

k_T = coefficient of permeability at the test temperature T.

μ_T = the viscosity of test water at the test temperature T.

μ_{20} = the viscosity of water at standard temperature(20^{o}C).

These correction factors are given at Table 3.1.

°C	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
10	1.3012	1.2976	1.2940	1.2903	1.2867	1.2831	1.2795	1.2759	1.2722	1.2686
11	1.2650	1.2615	1.2580	1.2545	1.2510	1.2476	1.2441	1.2406	1.2371	1.2336
12	1.2301	1.2268	1.2234	1.2201	1.2168	1.2135	1.2101	1.2068	1.2035	1.2001
13	1.1968	1.1936	1.1905	1.1873	1.1841	1.1810	1.1777	1.1746	1.1714	1.1683
14	1.1651	1.1621	1.1590	1.1560	1.1529	1.1499	1.1469	1.1438	1.1408	1.1377
15	1.1347	1.1318	1.1289	1.1260	1.1231	1.1202	1.1172	1.1143	1.1114	1.1085
16	1.1056	1.1028	1.0999	1.0971	1.0943	1.0915	1.0887	1.0859	1.0803	1.0802
17	1.0774	1.0747	1.0720	1.0693	1.0667	1.0640	1.0613	1.0586	1.0560	1.0533
18	1.0507	1.0480	1.0454	1.0429	1.0403	1.0377	1.0351	1.0325	1.0300	1.0274
19	1.0248	1.0223	1.0198	1.0174	1.0149	1.0124	1.0099	1.0074	1.0050	1.0025
20	1.0000	0.9976	0.9952	0.9928	0.9904	0.9881	0.9857	0.9833	0.9809	0.9785
21	0.9761	0.9738	0.9715	0.9692	0.9669	0.9646	0.9623	0.9600	0.9577	0.9554
22	0.9531	0.9509	0.9487	0.9465	0.9443	0.9421	0.9399	0.9377	0.9355	0.9333
23	0.9311	0.9290	0.9268	0.9247	0.9225	0.9204	0.9183	0.9161	0.9140	0.9118
24	0.9097	0.9077	0.9056	0.9036	0.9015	0.8995	0.8975	0.8954	0.8934	0.9813
25	0.8893	0.8873	0.8853	0.8833	0.8813	0.8794	0.8774	0.8754	0.8734	0.8714
26	0.8694	0.8675	0.8656	0.8636	0.8617	0.8598	0.8579	0.8560	0.8540	0.8521
27	0.8502	0.8484	0.8465	0.8447	0.8428	0.8410	0.8392	0.8373	0.8355	0.8336
28	0.8318	0.8300	0.8282	0.8264	0.8246	0.8229	0.8211	0.8193	0.8175	0.8157
29	0.8139	0.8122	0.8105	0.8087	0.8070	0.8053	0.8036	0.8019	0.8001	0.7984
30	0.7967	0.7950	0.7934	0.7917	0.7901	0.7884	0.7867	0.7851	0.7834	0.7818
31	0.7801	0.7785	0.7769	0.7753	0.7737	0.7721	0.7705	0.7689	0.7673	0.7657
32	0.7641	0.7626	0.7610	0.7595	0.7579	0.7564	0.7548	0.7533	0.7517	0.7502
33	0.7486	0.7471	0.7456	0.7440	0.7425	0.7410	0.7395	0.7380	0.7364	0.7349
34	0.7334	0.7320	0.7305	0.7291	0.7276	0.7262	0.7247	0.7233	0.7218	0.7204
35	0.7189	0.7175	0.7161	0.7147	0.7133	0.7120	0.7106	0.7092	0.7078	0.7064

TABLE 3.1 VISCOSITY CORRECTIONS FOR μ_T / μ_{20}

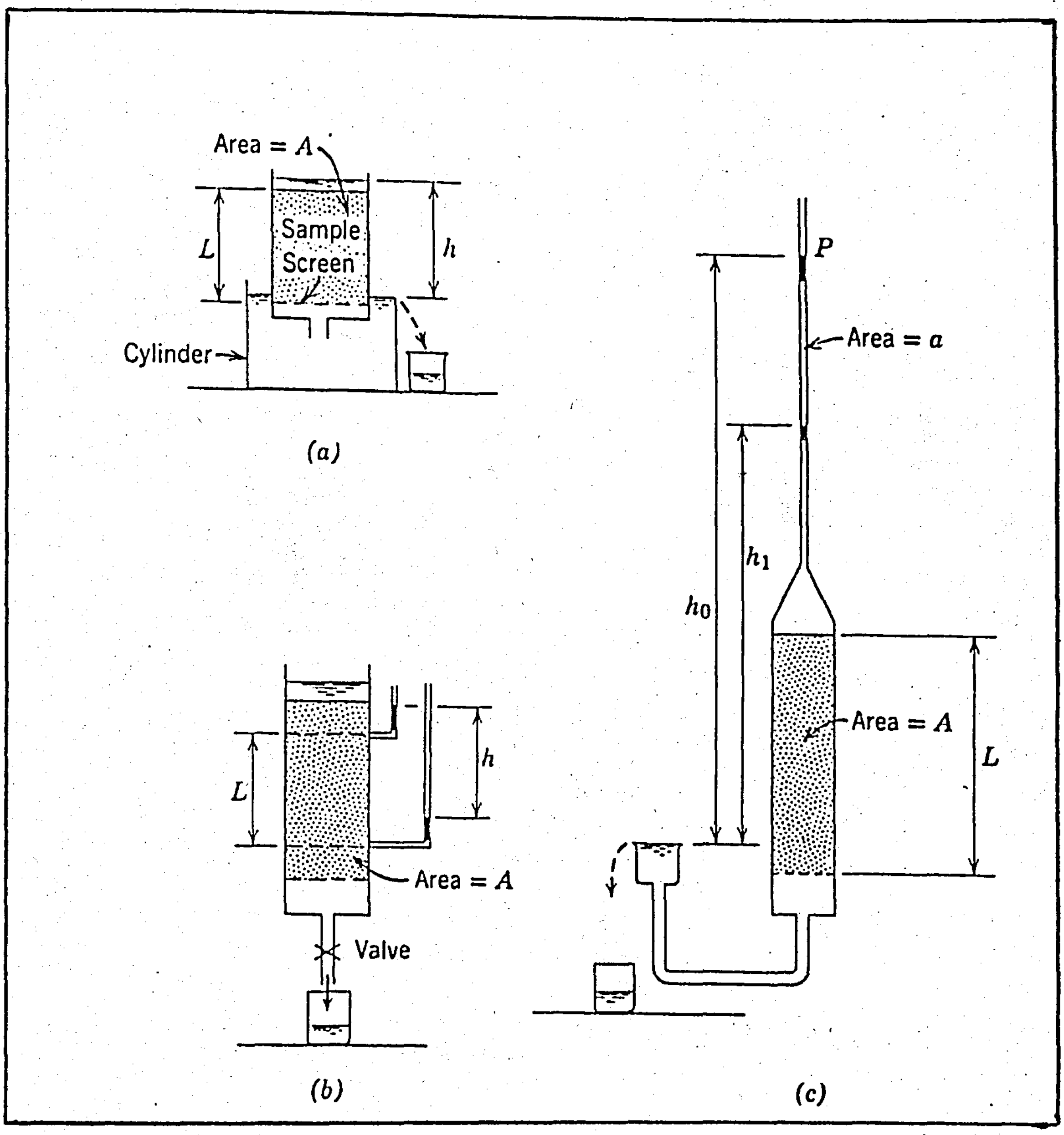


FIG.3.5 LABORATORY PERMEAMETERS

a. Constant Head Permeameter.

b. Constant Head Permeameter.
(The arrangement here eliminates due to filter skin at top or bottom of specimen).

c. Falling Head Permeameter. (Terzaghi and Peck, 1944)

3.6- FIELD METHODS FOR DETERMINING PERMEABILITY

No matter now carefully laboratory tests are made, they represent only minute volumes of soil at individual points in large masses. Their value in solving field seepage and drainage problems depends on how well they represent masses of materials that actually exist in the field. When used with careful consideration of field conditions, laboratory methods can be of considerable value. Nevertheless, in important projects it is often advisable to require field tests that measure the permeabilities of large masses of soil in situ.

3.6.1 Well-Pumping Test. Steady State.

A widely used field permeability test is the well-pumping test, in which water is pumped into or out of a while water readings are made in several nearly sounding wells. The test is continued until steady conditions are reached.

Performing this test in situ, the coefficient of permeability can be computed by using the following equation:

$$k = \frac{2.3\ q}{\pi(\ h_2^2 - h_1^2\ )} \log \frac{r_2}{r_1} \qquad \ldots(3.11)$$

Eqn. 3.11 is based on the following assumptions:

1. The pumping well penetrates the full thickness of water bearing formation.
2. A steady-state flow condition exists.
3. The water bearing formation is homogenous and isotropic and extends an infinite distance in all directions.

4. The hydraulic gradient at any point is a constant from the top to the bottom of the water-bearing layer and is equal to the slope of the water surface.

A typical arrangement for Well-Pumping Test is given at FIG. 3.6.

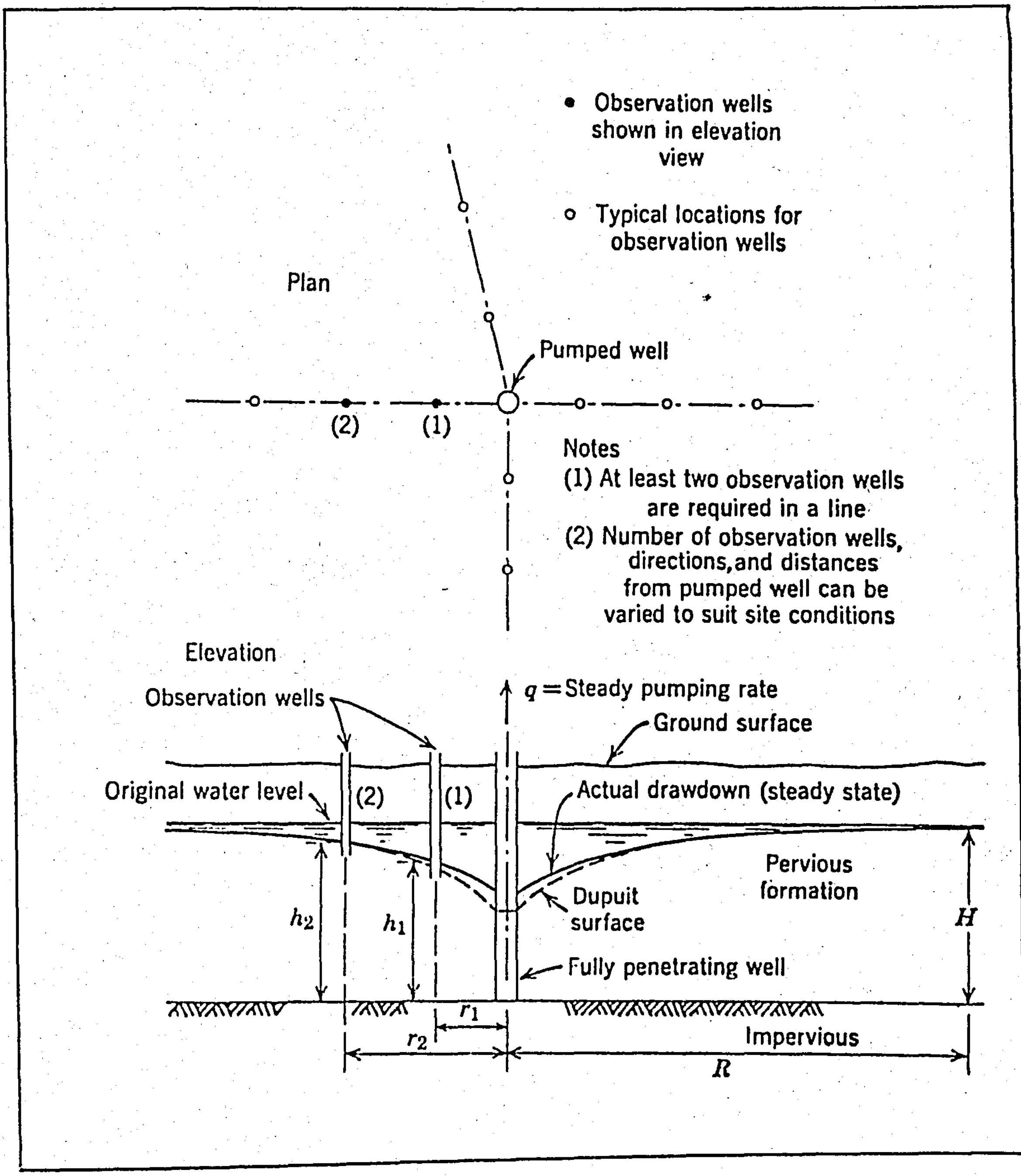


FIG 3.6 TYPICAL ARRANGEMENTS FOR DETERMINING PERMEABILITY BY WELL- PUMPING TEST

3.6.2 Pumped Wells With Observation Holes.Nonsteady State.

When field permeability tests are made with the method described for steady state flow,pumping must be continued until the water levels in observation holes have approximately stabilized.Although true equilibrium may require extremely long periods of pumping,practical results usually can be obtained by pumping at a steady rate for periods that range from a few hours to a few days,depending largely on the permeability.

During the period in which the water table around a pumped well is lowering water is draining out of the aquifer. Useful solutions to seepage conditions during the nonsteady period are furnished by basic differential equations.

Performing this test in situ,the coefficient of permeability can be computed by using the following equation:

$$k = \frac{q}{4\pi D(S_2 - S_1)} \ln \frac{t_2}{t_1} \qquad \ldots(3.12)$$

where,

k= coefficient of permeability in feet per day.

D= the original thickness of the aquifer in feet.

S_1, S_2=readings of drawdowns in feet.

t_1, t_2=reading time in second.

q= rate of flow in cu ft per day.

3.6.3 Borehole Tests

Because complete well-pumping tests are costly, efforts are frequently made to estimate permeabilities of inplace soils and rocks by pumping into or out of drill holes without the use of observation wells. The procedures are used in exploration boreholes since they provide a physical index of the flow into or moderate volumes of inplace material at relatively little cost. They can furnish useful permeability information, but they must be applied with care because the results are not easily checked for accuracy and errors are possible.

The most frequent causes of errers are the following:

1. Leakage along casing and around packers.

2. Clogging due to sloughing of fines or sediment in the test water.

3. Air locking due to gas bubbles in soil or water.

4. Flow of water into cracks in soft rocks that are opened by excessive head in test holes.

3.6.3.1 Open - end Tests

If the hole extends below the groundwater level, it should be kept filled with water to minimize the squeezing of soil into the bottom of the casing. The test is made by maintaining a constant head by adding clear water through a measuring device. When tests are made above the water table a smooth consistent water level is seldom obtained, and surging a few tenths of a foot at a steady rate of flow for about 5 minute is considered a satisfactory test.

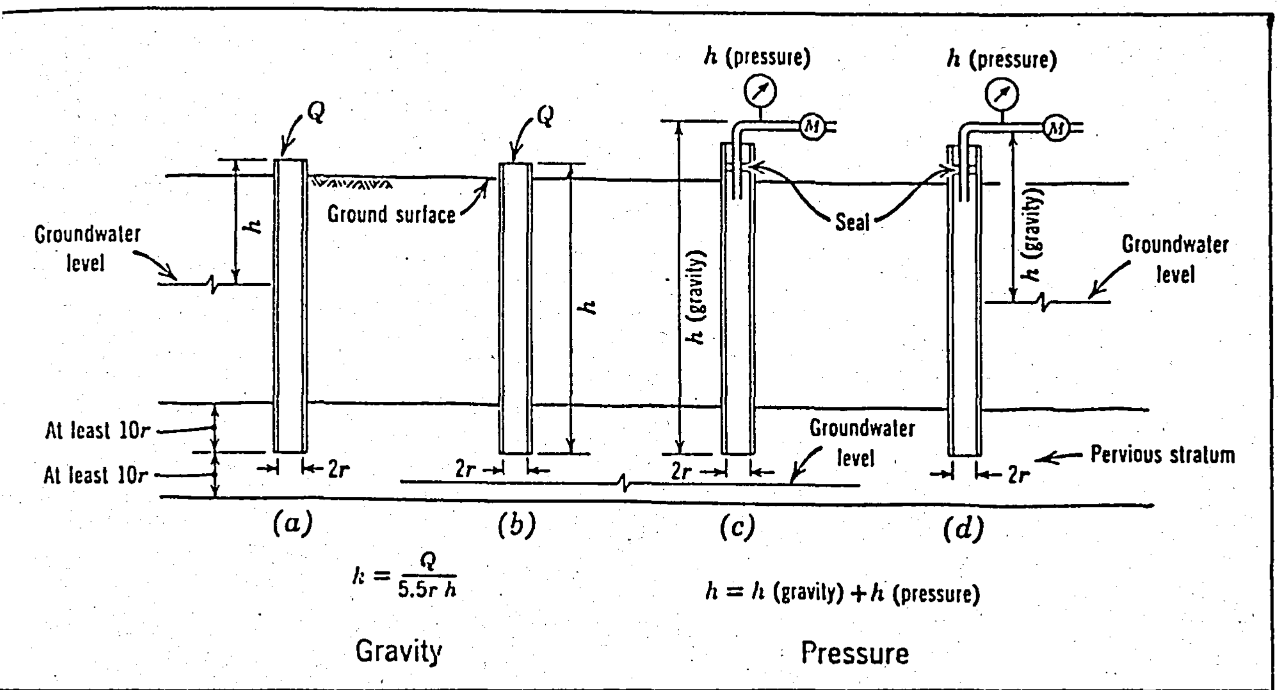


FIG.3.7 AN OPEN-END TEST FOR SOIL PERMEABILITY WHICH CAN BE MADE IN THE FIELD (Cedergren, 1977).

When desired, additional pressure can be added to the gravity head. The permeability is calculated from the following relationship determined by Electric Analogy Tests:

$$k = \frac{q}{5.5\ r\ h} \qquad \ldots(3.13)$$

where,

q = the constant rate of flow into the hole.

r = the inside radius of casing.

h = the differential head of water used in maintaining the steady rate.

or, according to The Bureau of Reclamation,

$$k = C_1 \frac{q}{h} \qquad \ldots (3.14)$$

where,

C_1 = factor varying with the size of casing (generally between 102,000 and 204,000).

q = the constant rate of flow into the hole in gallons/min.

h = the differential head of water used in maintaining the steady rate in feet.

3.6.3.2 Packer Tests

If the formation is strong enough to remain open, tests can be made above or below the water table. These tests are used for testing bedrock with the number of packers necessary to isolate the section of hole being tested. Permeabilities can be calculated from the following relationships:

$$k = \frac{q}{2\pi L\, h} \ln \frac{L}{r} \quad , \quad L \gg 10r \qquad \ldots (3.15)$$

$$k = \frac{q}{2\pi L\, h} \sinh^{-1} \frac{L}{2\,r} \quad , \quad 10r > L \gg r \qquad \ldots (3.16)$$

where,

k = the coefficient of permeability of soil.

q = the constant rate of flow into the hole.

L = the length of the section of hole being tested.

h = the differential head.

r = the radius of the hole.

or, according to The Bureau of Reclamation,

$$k = C_p \frac{q}{h} \qquad \ldots(3.17)$$

where,

k = coefficient of permeability in feet per year.

C_p = factor varying with the size of the test hole, and the length of test section.

q = the constant rate of flow into the hole in gallons/min.

h = the differential head.

C_p varies from 31,000 to 2,800 with the size of the test hole,and the length of the test section.

3.6.4 Tube Method of Determining Coefficient of Permeability

A simple procedure,called the " Tube Method " may be used for measuring the permeability of soil in situ below a shallow water table.In this method,a tube of known diameter is placed tightly in a hole of the same size to a known depth below a water table.Then the water is pumped out to some known elevation below the water table and above the bottom of the tube; and water from the surrounding soil is allowed to flow into tube through the bottom.The rise of water level in a measured period of time is observed,and the permeability is computed by means of the following relationship:(Spangler,1966)

$$k = \frac{A}{Et_1} \ln \frac{h_o}{h_1} \qquad \ldots(3.18)$$

where,

k= coefficient of permeability of tested soil.

A= area of tube.

h_o= distance from water table to water level in tube at beginning of test.

h_1= distance from water table to waterlevel in tube at end of test.

t_1=elapsed time within which distance from water table to water level decreases from h_o to h_1.

E = the E-factor,which is a coefficient.(See Table 3.2)

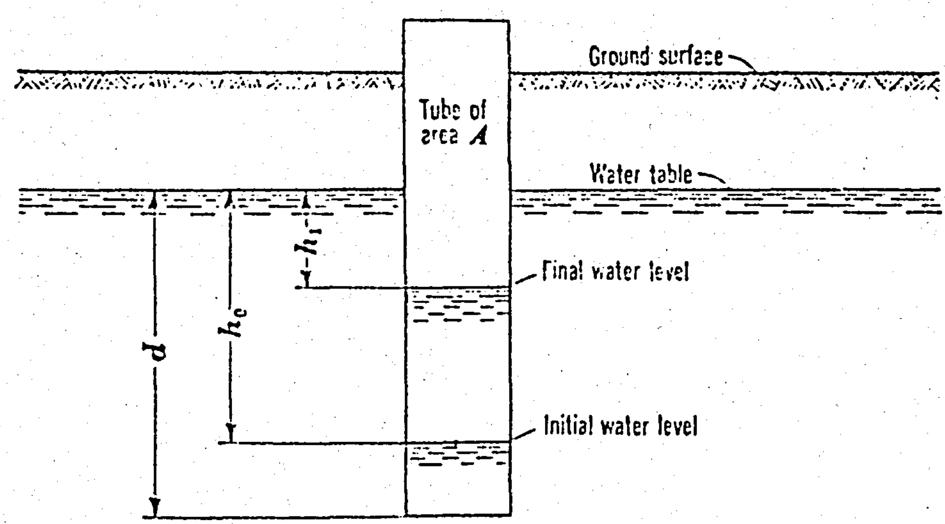


FIG.3.8 TUBE METHOD OF DETERMINING COEFFICIENT OF PERMEABILITY (Spangler,1966).

Depth / Diameter	Diameter of Tube, Inches						
	1	2	3	4	5	6	8
1	....	....	....	....	....	15.6	20.9
2	....	....	....	....	13.1	15.5	20.8
3	....	....	....	10.3	13.0	15.5	20.7
4	....	....	7.7	10.3	12.9	15.4	20.5
5	....	....	7.7	10.2	12.9	15.3	20.4
6	....	5.1	7.6	10.2	12.8	15.2	20.3
7	....	5.1	7.6	10.1	12.7	15.2	20.2
8	....	5.1	7.5	10.1	12.7	15.1	20.1
10	....	5.0	7.5	9.9	12.5	14.9	....
12	2.5	5.0	7.4	9.8	12.4	....	....
15	2.4	4.9	7.2	9.7	....	....	....
25	2.3	4.6	6.8	....	....	....	....
40	2.1	4.0	....	....	....	....	....
60	1.9	....	....	....	....	....	....
100	1.5	....	....	....	....	....	....

TABLE 3.2. VALUES OF E - FACTOR -(in inch units.) (Spangler,1966).

3.6.5 Piezometer Method of Determining Coefficient of Permeability

For measurements of permeability at greater depths, a thin-walled electrical conduit having an inside diameter of 1 inch may be used. This method is called the Piozemeter Method. The pipe is driven a short distance into the soil , a soil auger with a diameter of 15/16 in. is bored through the pipe and into the soil to a depth of 4 inches below the bottom and the soil is removed. The pipe is then driven into the soil 4 inches, and the augering is repeated. This process is continued with the bottom of pipe has reached the desired depth, there being finally a space 4 inches deep below the bottom of the pipe. This method of driving the pipe prevents compaction of the soil sample to be tested. The formula for determining the permeability coefficient by this procedure is as follows:

$$k = \frac{A}{Et_1} \ln \frac{h_o}{h_1} \qquad \ldots(3.18)$$

where,

k = coefficient of permeability of tested soil.

A = area of the piezometer.

h_o = distance from water table to water level in piezometer at beginning of test.

h_1 = distance from water table to water level in piezometer at the end of the test.

t_1 = elapsed time within which distance from water table to water level decreases from h_o to h_1.

E = the E-factor, which is a coefficient. (see Fig. 3.10)

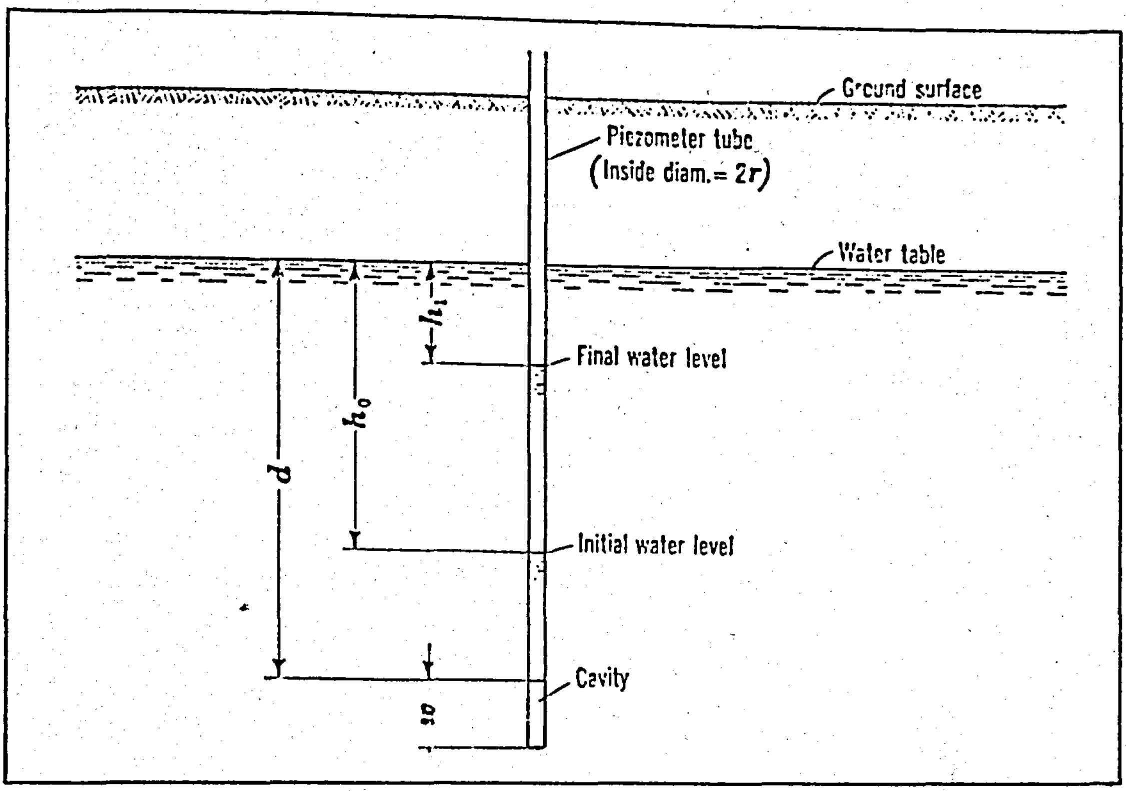


FIG. 3.9 PIEZOMETER METHOD FOR DETERMINING COEFFICIENT OF PERMEABILITY (Spangler,1966)

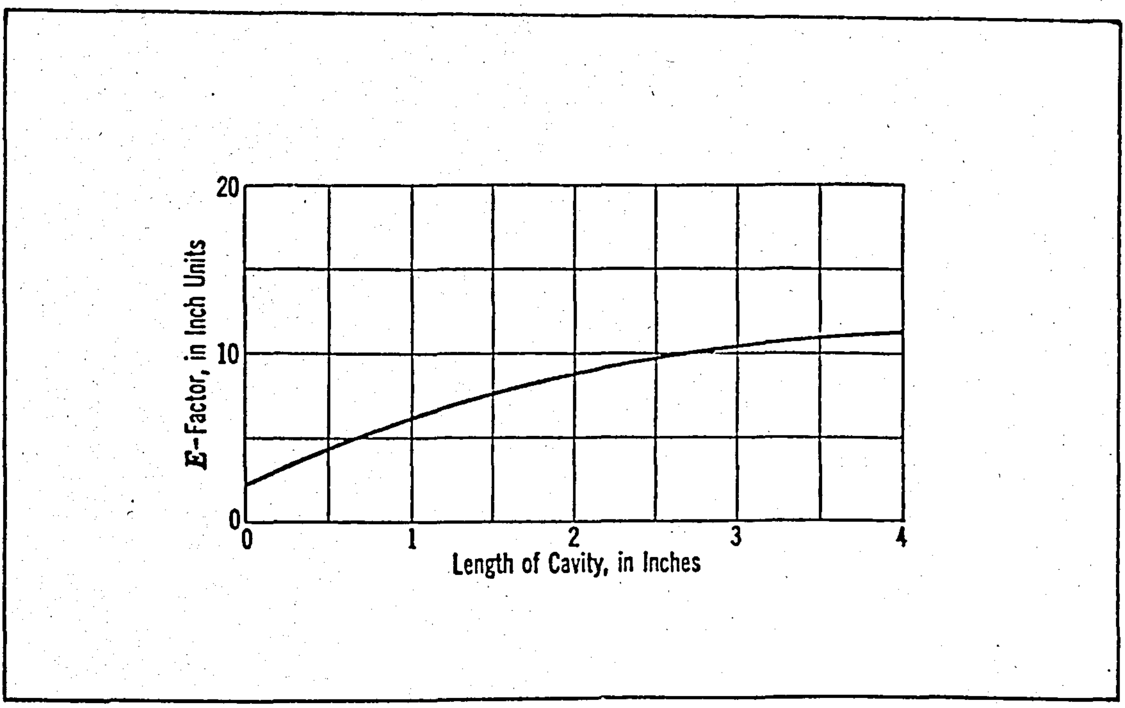


FIG. 3.10 VALUES OF E-FACTOR FOR CAVITIES 1 INCH IN DIAMETER

3.6.6 Auger Hole Method of Determining Coefficient of Permeability

In these method, it is advisable to pump water from the hole to refill, in order to flush out the soil pores at its sides. If the auger hole extends completely through a pervious stratum to an impervious layer, the flow situation is subject to exact mathematical analysis. However, the results can be used to obtain a good approximation, even in the absence of an impervious layer, if the ratio of the depth of the auger hole to its diameter is large. The formula for determining the permeability coefficient by this procedure is as follows:

$$k = 0.617 \frac{r}{Sd} \frac{dh}{dt} \qquad \ldots(3.19)$$

where,

k = coefficient of permeability of tested soil.

S = a coefficient which is dependent on the ratios $\frac{h}{d}$ and $\frac{r}{d}$ (see Fig. 3.12)

d = depth of hole below water table.

$\frac{dh}{dt}$ = rate of rise of water level in hole at depth h.

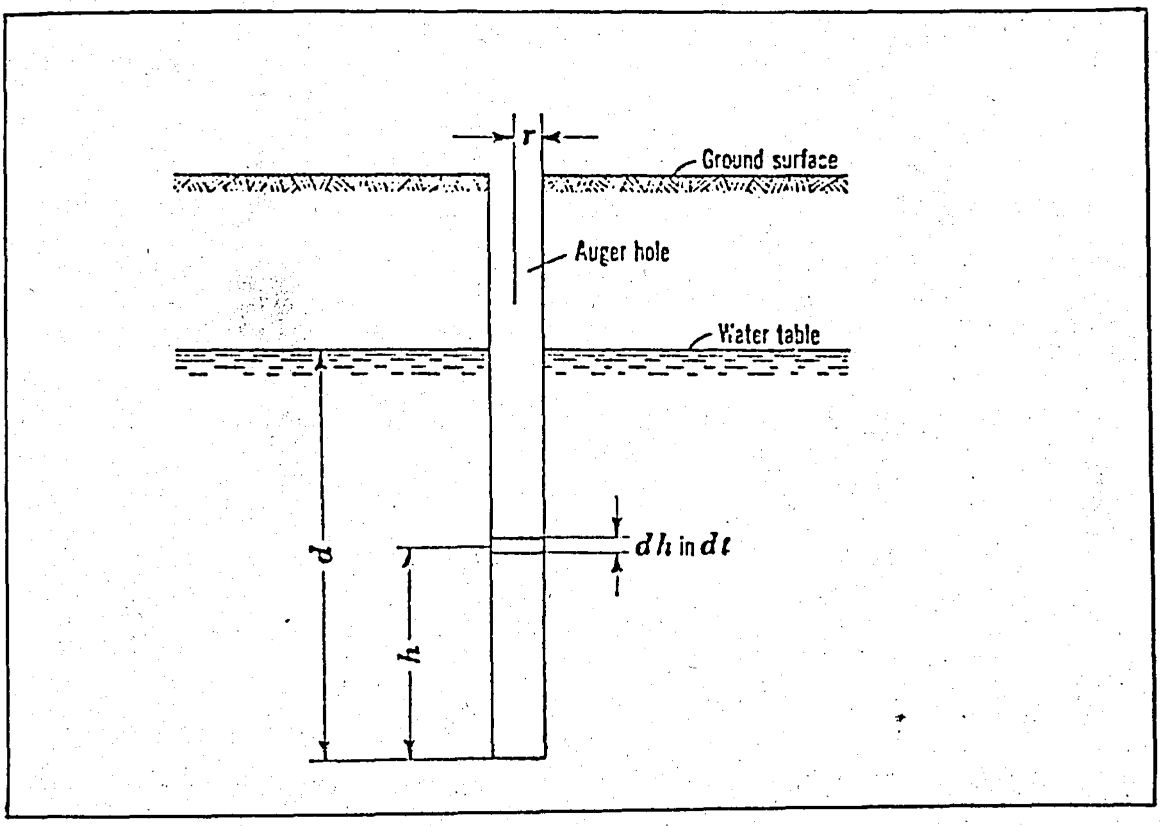


FIG. 3.11 AUGER-HOLE METHOD FOR DETERMINING COEFFICIENT OF PERMEABILITY (Spangler,1966)

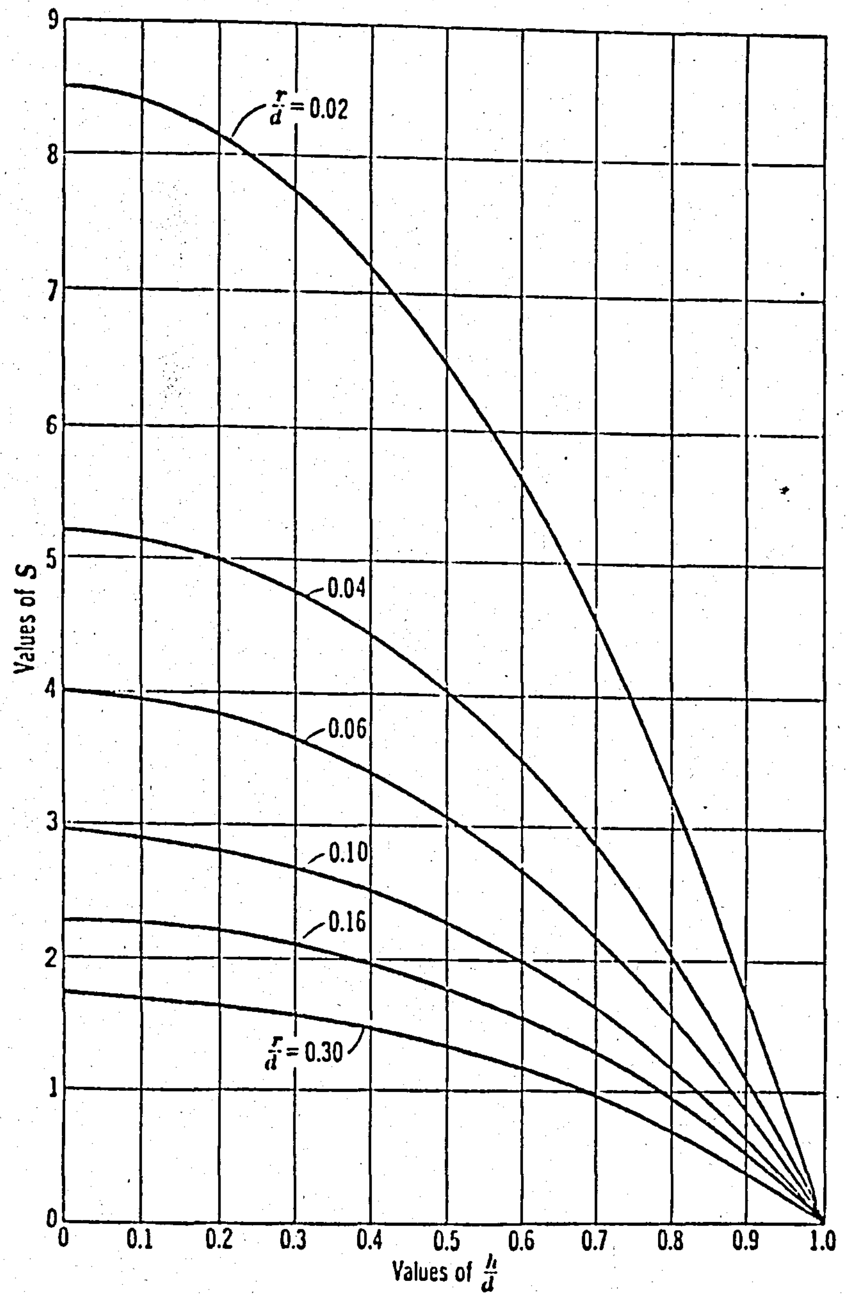


FIG. 3.12 VALUES OF S in eqn.(3.19) (Kirkham,1954).

3.7- FIELD METHODS THAT DEPEND ON SEEPAGE VELOCITIES

When at a common point, the velocity of the flowing water and the hydraulic gradient are known, the permeability can be estimated from the following relationship:

$$k = \frac{V_s \, n_e}{i} \qquad \ldots(3.3)$$

where,

k = the coefficient of permeability.

V_s = the average seepage velocity.

n_e = the effective porosity.

i = the hydraulic gradient.

Frequently the hydraulic gradient of an existing water table can be estimated from wells in the area. If not observation wells must be installed. The velocity of flow can be determined by a number of practical methods.

An electrolyte or radiactive charge is inserted into the sloping water table in hole A, the time for the charge to reach hole B is measured with suitable instruments, and the seepage velocity is determined by dividing L by time t. The effective porosity, n_e, is determined from test data for the in-place soil; if no tests are available, it is estimated, and then the permeability is calculated from Eqn. (3.3).

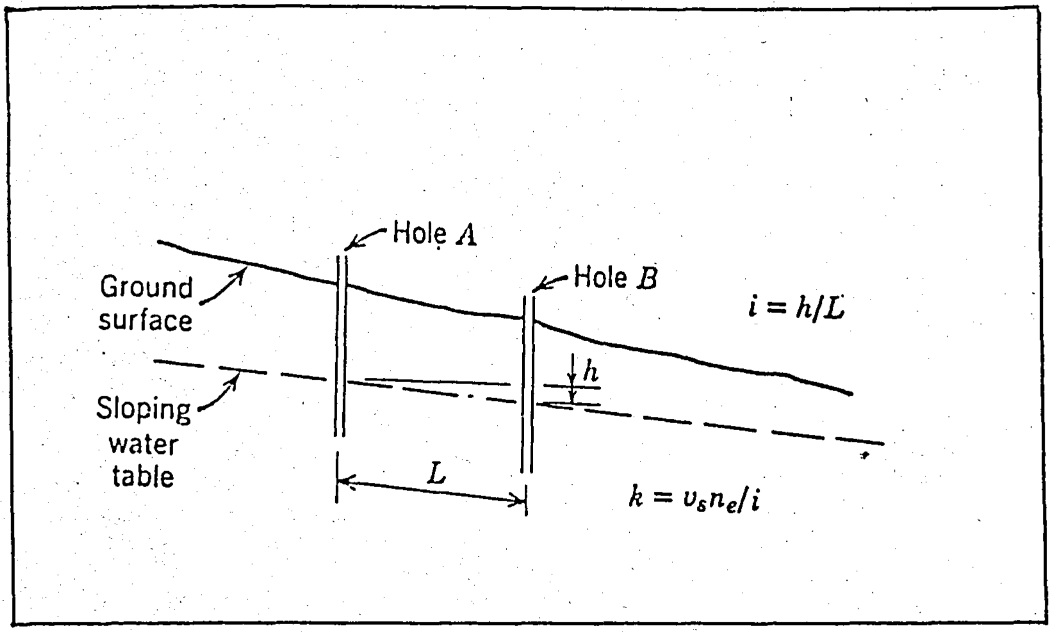


FIG. 3.13 TYPICAL ARRANGEMENT FOR DETERMINING SOIL PERMEABILITY BY MEASUREMENT OF SEEPAGE VELOCITY

3.8- FIELD METHODS THAT DEPEND ON OBSERVATION OF SPREADING OR RECEDING GROUNDWATER MOUNDS

In this method, computations are based on measurements of the rate of spread of saturation or, volumes of water flowing into or out of soil systems, estimated from volumes of soil that become saturated or unsaturated during a known period of time.

3.8.1 Estimating k from Rate of Spread of Water. Unconfined Flow

When water is spreading from a sudden rise of rivers in flood stage, the mass permeabilities of soil formations can often be estimated from the spread and rise of saturation measured in observation wells. If the spreading takes place in relatively uniform soil with unconfined flow, an estimate of permeabilities can be made with the use of transient flow nets. If the time of spread is known, permeability can be estimated (Cedergren, 1977):

$$k = \frac{V_s\, n_e}{i} \qquad \ldots(3.3)$$

If the travel time is known, permeability can be estimated:

$$k = \frac{n_e}{T} \sum \frac{\Delta l}{i} \qquad \ldots(3.19)$$

where,

k = coefficient of permeability.

l = increment of distance.

T = the total time.

i = the hldraulic gradient.

3.8.2 Estimating k from Quantity of Water Flowing Into or Out of Soil

When water is spreading from a rapidly rising river through highly permeable strata and rising into moderately permeable upper strata, the permeability of the underlying strata often can be estimated from the rise of water in observation wells. Practical estimates of permeability can be obtained by the following procedure:

1.A cross section is plotted to show the initial position of the water table and its position after the river has been at level 2 (See Fig.3.14) for T days.

2.Mean travel distance L is estimated.(L is the distance to the center of gravity of the saturated area).

Then k is calculated from Darcy's Law:

$$k = \frac{q}{i\ A} \qquad \ldots(3.20)$$

where,

k = coefficient of permeability in feet/day.

q = the rate of flow in cu ft/day.

i = the hydraulic gradient.

A = the cross-sectional area perpendicular to the flow direction in sq ft/1 in.

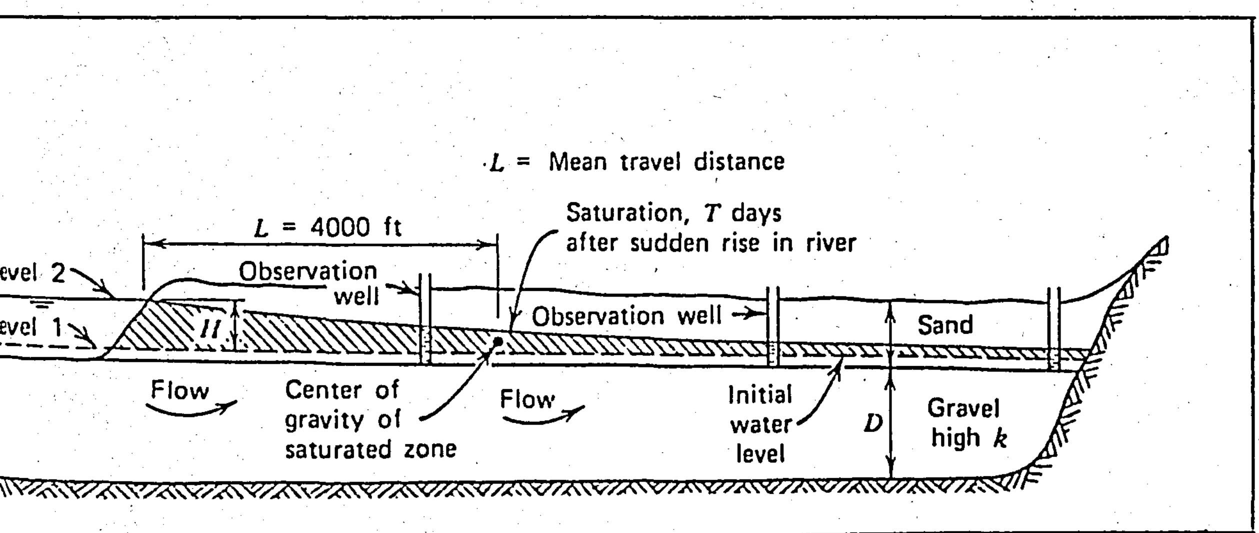


FIG. 3.14 ESTIMATING k FROM QUANTITY OF WATER FLOWING INTO A SOIL SYSTEM (Cedergren,1977).

CHAPTER 4

FILTRATION AND DRAINAGE

4.1- BASIC REQUIREMENTS OF FILTERS AND DRAINS

The process by which percolating water or groundwater is removed from soils and rocks by natural or artificial means is called " drainage ".When an analysis is being made of the best means of controlling water in engineering works, it is important to try to identify the sources of the water.In some cases,it may be possible to reduce or entirely cut off the inflows by means of seepage-reducing methods such as blankets, linings,cutoffs,and grout curtains.In most cases,however,the safest,most economical and satisfactory solution is achieved by drainage systems.Of great importance in drainage design is the need for developing systems capable of removing all the water reaches them without excessive head build-up and without clogging or piping.Furthermore,designers should analyze every component of a drainage system(filters,conducting layers, collectors,outlets) to ensure that the entire system will have the necessary capacity and will function as intended.

To some degree porous wicks of cloth or paper,fiber-glass blankets,and other manufactured products are used as filter and drain materials,but the predominating drainage material is porous mineral aggregate.Good quality aggregates are virtually indestructible,relatively incompressible,readily available in most areas,and relatively inexpensive.When used correctly,porous drainage aggregates can have a vital part in the permanent performance of a great many kinds of civil engineering works,They are frequently used in drainage systems in conjunction with slotted,jointed,or porous pipes, which assist in the collection and removal of seepage.

In recent years,synthetic filter fabrics,geotextiles,have taken an increased importance in drainage systems.Filter fabrics are used primarily as a substitute for a fine aggregate filter in regions in which good quality aggregates are scarce or in situations in which a filter fabric may be easier to install than a fine aggregate filter.

Filters and drains can provide permanent security against damaging actions of seepage and groundwater,however, certain fundamental requirements must be strictly enforced.If filters and drains are to serve their intended purpose,the materials used in their construction must have the correct gradation, and they must be handled and placed with care to contamination and segregation.

Many of the problems associated with the design of adequate filters and drains stem from the need for satisfying two conflicting requirements.

1.Piping Requirement.The pore spaces in drains and filters that are in contact with erodible soils and rocks must be small enough to prevent particles from being washed in or through them.

2. Permeability Requirement. The pore spaces in drains and filters must be large enough to impart sufficient permeability to permit seepage to escape freely and thus provide a high degree of control over seepage forces and hydrostatic pressures.

4.2- PREVENTION OF PIPING

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 often are exhibited by sink holes that form in arid and semiarid lands when fine sand, silt, loess, and clay wash into subterranean tubes or cracks.

Many engineering works produce large hydraulic gradients that are conducive to piping. When sewers are constructed below water table in erodible sand or silt, joints must be meticulously sealed; otherwise serious infiltration is likely to occur. Piping is a common cause of failure in overflow weirs, earth dams, reservoirs, and other hydraulic structures. Whenever filters and drains are required for the control of seepage and ground water in relation to structures, they should have a high degree of resistance to piping.

4.2.1 Grading of Drainage Aggregates To Control Piping

To prevent the movement of erodible soils and rocks into or through filters, the pore spaces between the filter particles should be small enough to hold some of the larger particles of protected materials in place. If three perfect spheres have diameters greater than six and one-half times the diameter of a smaller sphere, the smaller spheres can move through the larger. (See Fig.4.1). Soils and aggregates are always composed of ranges of particle sizes, and if the pore spaces in filters are small enough to hold the 85% size (D_{85}) of adjacent soils in place the finer soil particles will also be held in place.

Bertram (1940) gives Criteria for filter design as:

$$\frac{D_{15}\ \text{filter}}{D_{85}\ \text{soil}} < 4 \text{ to } 5 < \frac{D_{15}\ \text{filter}}{D_{15}\ \text{soil}} \qquad \ldots(4.1)$$

where,

D_{15} filter = 15% size of filter material.

D_{85} soil = 85% size of protected soil.

D_{15} soil = 15% size of protected soil.

Criterion 1. The 15% size (D_{15}) of afilter material must be not more than four or five times the 85% (D_{85}) of a protected soil. The ratio of D_{15} of a filter to D_{85} of a soil is called " The Piping Ratio ".

Criterion 2. The 15% (D_{15}) of a filter material should be at least four or five times the 15% (D_{15}) of a protected soil.

To prevent the movement of soil particles into or through filters the U.S. Army Corps of Engineers (1985) require that the following conditions be satisfied:

$$\frac{D_{15}\ \text{filter}}{D_{85}\ \text{soil}} \leqslant 5 \qquad \ldots(4.2)$$

and

$$\frac{D_{50}\ \text{filter}}{D_{50}\ \text{soil}} \leqslant 25 \qquad \ldots(4.3)$$

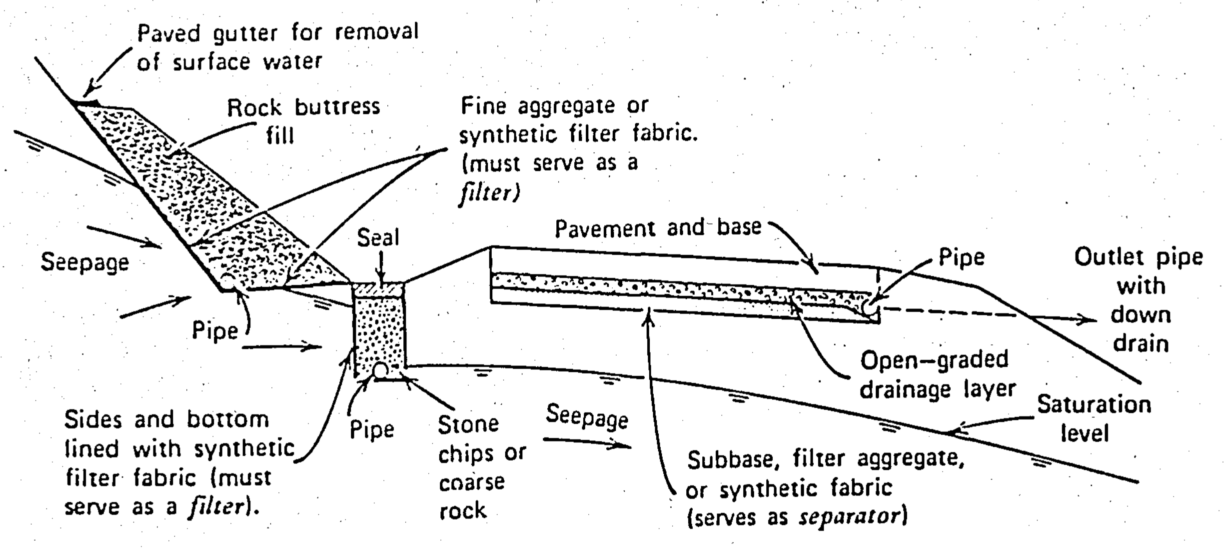
where,

D_{15} filter = 15% size of filter material.
D_{85} soil = 85% size of protected soil.
D_{50} filter = 50% size of filter material.
D_{50} soil = 50% size of protected soil.

The above criteria will be used when protecting all soils except for medium to highly plastic clays without sand or silt partings, which by the above criteria may require multiple-stage filters. For these clays, the D_{15} size of the filter may be as great as 0.4 mm. and the above D_{50} criteria will be disregarded. This relaxation in criteria for protecting medium to highly plastic clays will allow the use of a one-stage filter material however, the filter must be well-graded, and to insure non segregation of the filter material, a coefficient of uniformity (ratio of D_{60} to D_{10}) of not greater than 20 will be required. (Cedergren, 1977).

Piping failures are likely to occur in certain types of which erode by a process called "dispersion" or "defloculation". When the clay mass is in contact with water, individual clay particles are detached from the surface progressively and go into suspension. If water is flowing, the dispersed particles carried away and erosion channel or pipes can form quite rapidly. Frequently the initial flow of water is along one or more cracks caused by drying shrinkage, unequal foundation settlement, and so on, or by " hydraulic fracturing".

The chances of piping failures in dams built on or with dispersive clay soils can be greatly reduced by providing sandy gravel filters for vertical and horizontal drains designed to collect the seepage while holding the erodible soil in place. The filter adjacent to the soil must be fine enough to hold the dispersed soil particles in place; hence two or more progressively coarser layers will be needed in such drains. Tests should be made to establish the safe piping ratio (D_{15} of filter / D_{85} of soil) for all projets requiring the use of dispersive clays.



FIG? 4.1 ILLUSTRATION OF FILTER AND SEPARATOR FUNCTIONS OF PROTECTIVE FILTERS FOR DRAINS

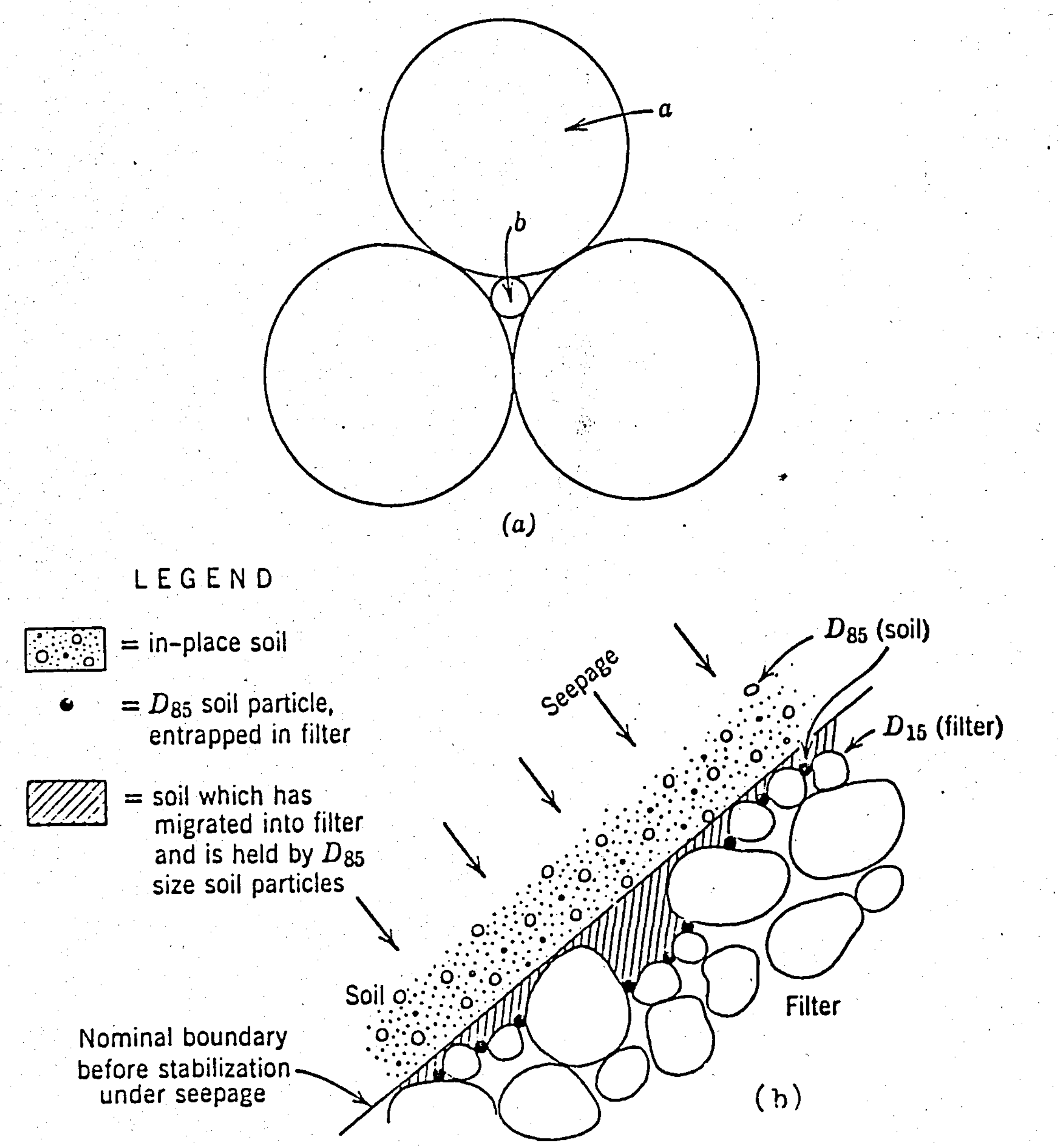


FIG. 4.2 ILLUSTRATION OF PREVENTION OF PIPING BY FILTERS

(a) Spherical particle b will just pass through pore space between three spheres six and one-half times the diameter of b.

(b) Conditions at a boundary between a soil and a protective filter.

4.2.2 Pipe Joints,Holes and Slots

When pipes are embadded in filters and drains,no unplugged ends should be allowed and the filter materials in contact with pipes must be coarse enough not to enter joints,holes or slots.

For slots,

$$\frac{D_{85}}{S} > 1.2 \qquad \ldots(4.4)$$

where,

D_{85} = 85% size of filter material.
S = slot width.

For circular holes,

$$\frac{D_{85}}{D_h} > 1.0 \qquad \ldots(4.5)$$

where,

D_{85} = 85% size of filter material.
D_h = hole diameter.

For openings in pipes,

$$\frac{D_{85}}{O_{max}} = 2 \qquad \ldots(4.6)$$

where,

D_{85} = 85% size of filter material.
O_{max} = maximum opening size of pipe drain.

4.3- EXAMPLES OF FILTER DESIGNS TO PREVENT PIPING

4.3.1 Rock Slope Protection

Frequently coarse rock is placed on the banks of leeves on the upstream faces of earthdams,and in other situations in which erodible soils must be protected from fast currents and wave action.If coarse rock is placed directly on fine soil, currents and waves may wash the soil out from under the rock and lead to undetermining and failure of expensive protective works or even to failure of the works being protected.

Soil erosion under rock slope protection can often be prevented by the placement of a filter layer of intermediate sized material between the soil and the rock.Sometimes erosion can be prevented by the use of well-graded rock containing suitable fines which work to the bottom during placement.If a single layer of well-graded material or spalls between the soil and the rock is depended on for erosion prevention, the work must be carried out carefully to make sure that an unsegregated filter layer is provided;otherwise it is possible that undermining could occur under severe wave action or fast currents.

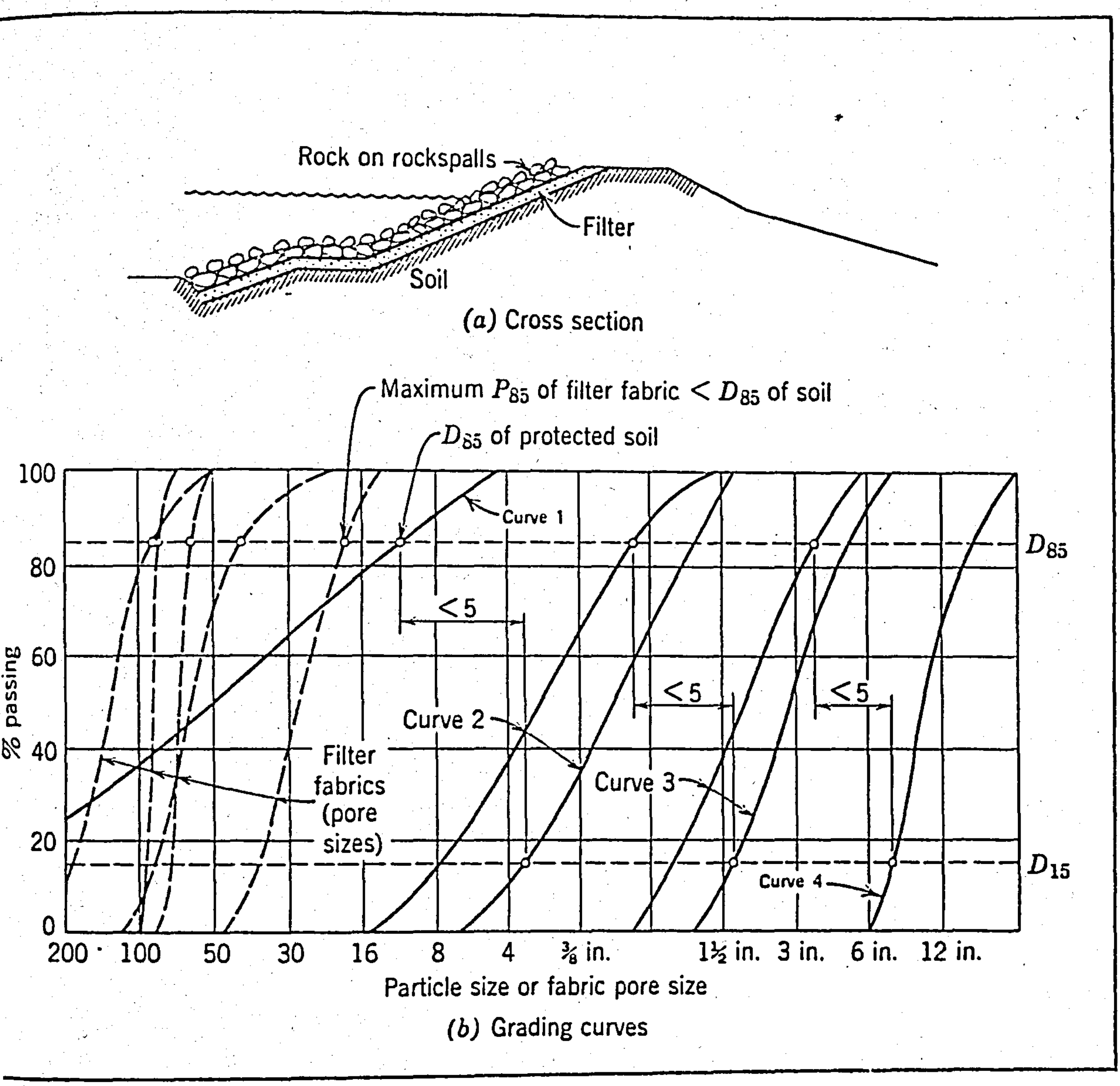


FIG.4.3 ROCK SLOPE DESIGNED TO PREVENT UNDERMINING.

Intermediate filter (curve 2) prevents erodible soil (curve 1) from washing through rock spalls (curve3) and through coarse rock (curve 4). Care must be taken to prevent segregation of the various courses. Alternate design replaces fine filter (curve 2) with a filter fabric.
(Cedergren,1977).

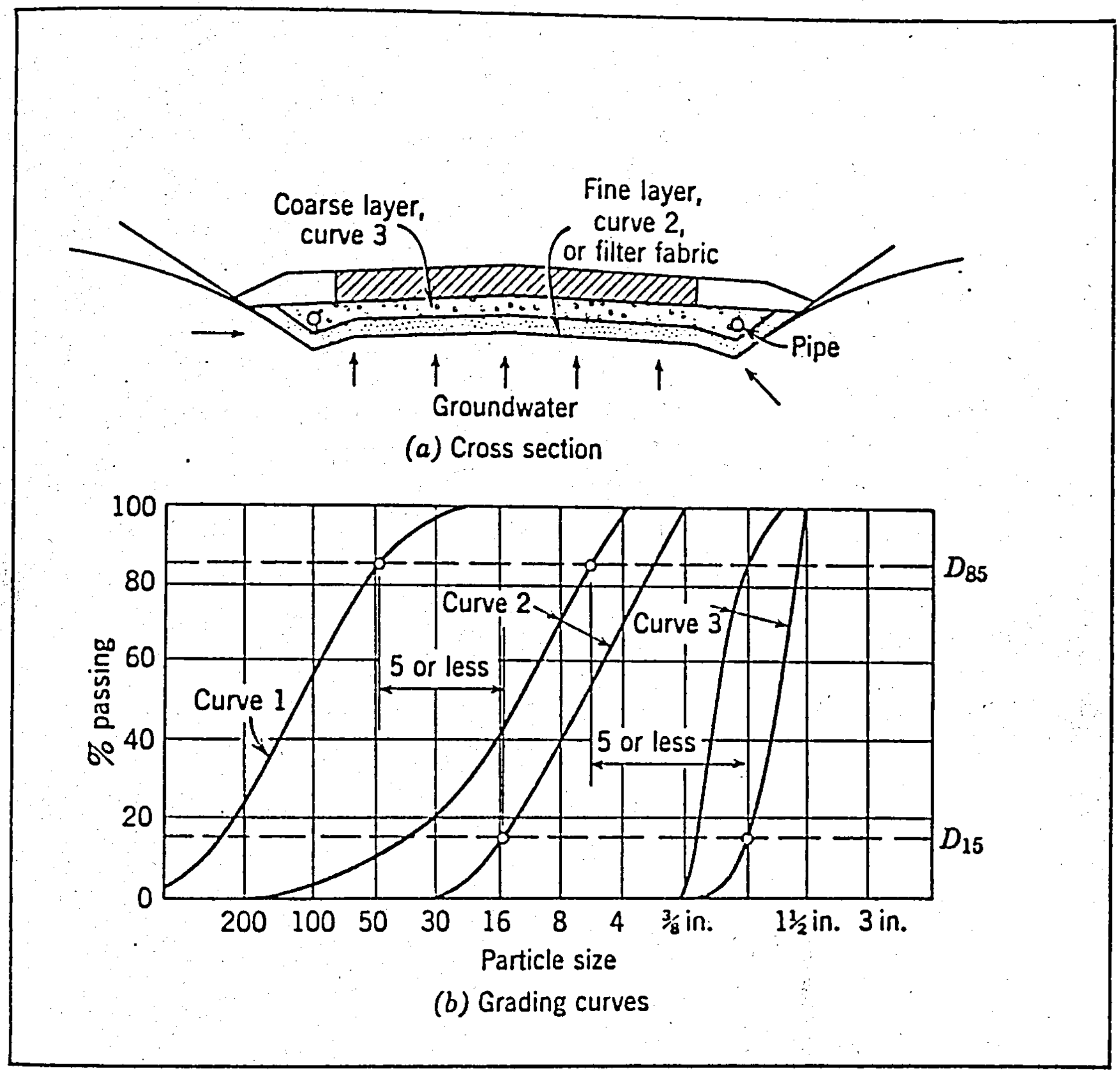


FIG. 4.4 DESIGN OF HIGHWAY ROADBED.

Fine filter (curve 2) prevents soil (curve 1) from pumping into open-graded drainage aggregate. (curve 3). (Cedergren,1977).

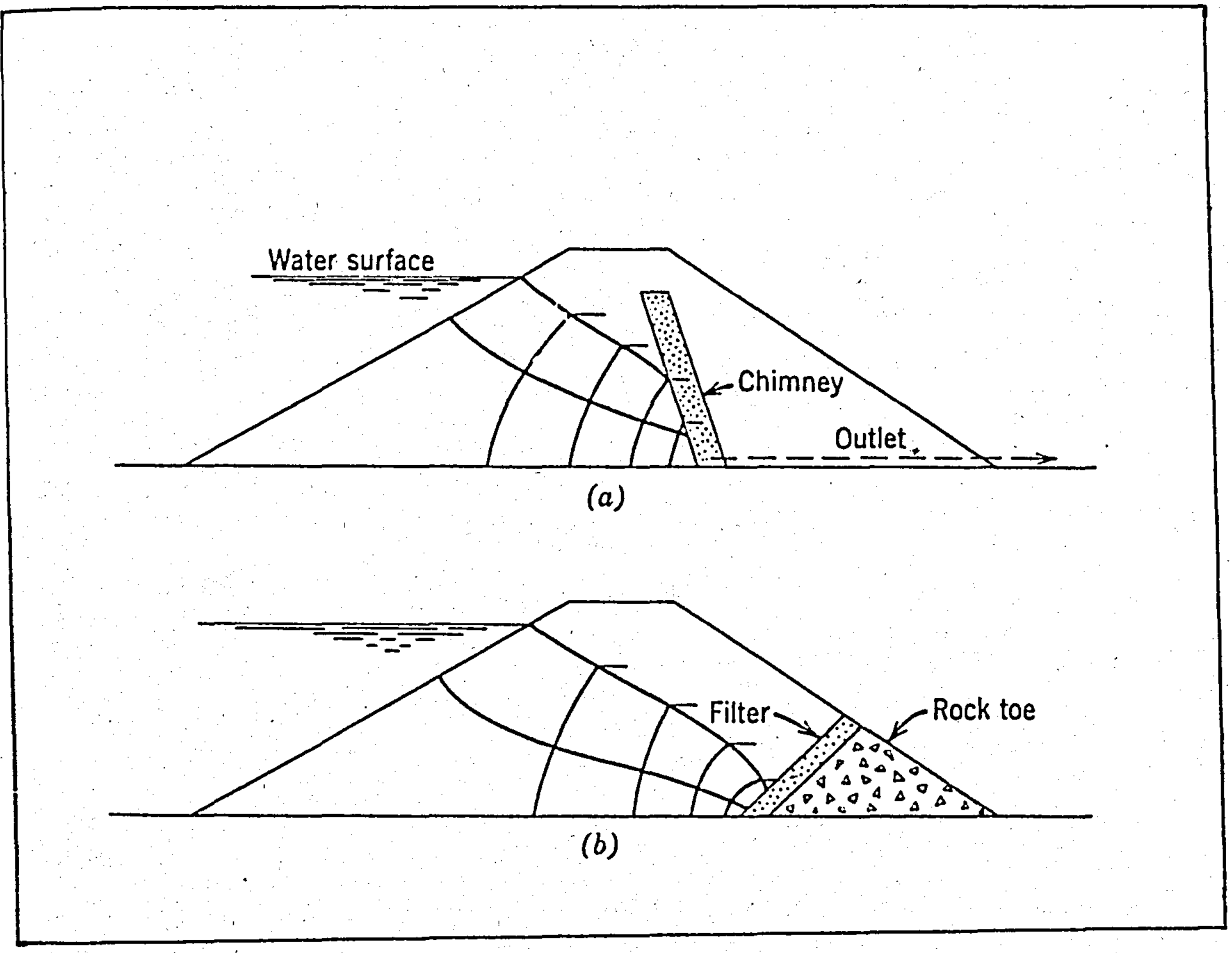


FIG. 4.5 EXAMPLES OF FILTERS USUALLY WORKING UNDER STEEP GRADIENTS TO REMOVE SEEPAGE.

(a) Dam with chimney drain.

(b) Dam with rock toe.

CHAPTER 5

USE OF GEOTEXTILES AS SOIL FILTERS

5.1- GEOTEXTILES AS SOIL FILTER

One of the major areas where geotextiles have found extensive use is as filters in one-directional drainage applications (e.g. subsurface drainage). For this application, geotextiles are used as replacements for graded granular filters because of their comparable performance, improved economy, consistent properties, and ease of placement.

Filter fabrics have two basic uses in drains for engineering works:

1. To serve as a true filter that must also act as a seperator to hold the soil in place and allow the free escape of water for long periods of time.

2. To serve as a separator or barrier to prevent the soil from mixing with a coarse aggregate layer when there is no significant long-time flow of water.

In all cases in which a fabric must serve as a true filter, it must have openings small enough to prevent more than minute amounts of the adjacent soil from passing through, but it must also have enough "open area" composed of sufficiently large openings to allowed unobstructed flow of water.

5.2- GENERAL CONSIDERATIONS FOR GEOTEXTILE FILTERS

Geotextile filter criteria have been developed and evaluated for general soil types involving relatively low hydraulic heads as occurs in subsurface drainage applications.There has been,however,no evaluation to determine the applicability of existing filter criteria to situations where large hydraulic heads are expected as is the case with internal filters in earth dams.

When a geotextile is placed to a soil and water is allowed to flow from the soil through the geotextile,a comlex interaction occurs between the soil particles and the pores in the geotextile.During an initial period immediately following the placement of the geotextile at the soil interface, the soil particles in the layer immediately adjacent to geotextile, which are smaller than the pores in the geotextile,migrate into and through the geotextile under the influence of soil water flow.Soil particles,which are larger than the pores in the geotextile and which lie immediately adjacent to it, orientate themselves against the upstream surface of the geotextile forming a bridging network.As soil water continues to pass through the geotextile,increasing amounts of the fine soil particles become trapped on this granular bridging network until such time as no soil particles can migrate across the boundaries of the geotextile.

A schematic diagram depicting the soil structure adjacent to the geotextile following completion of soil particle migration in Fig.5.1.

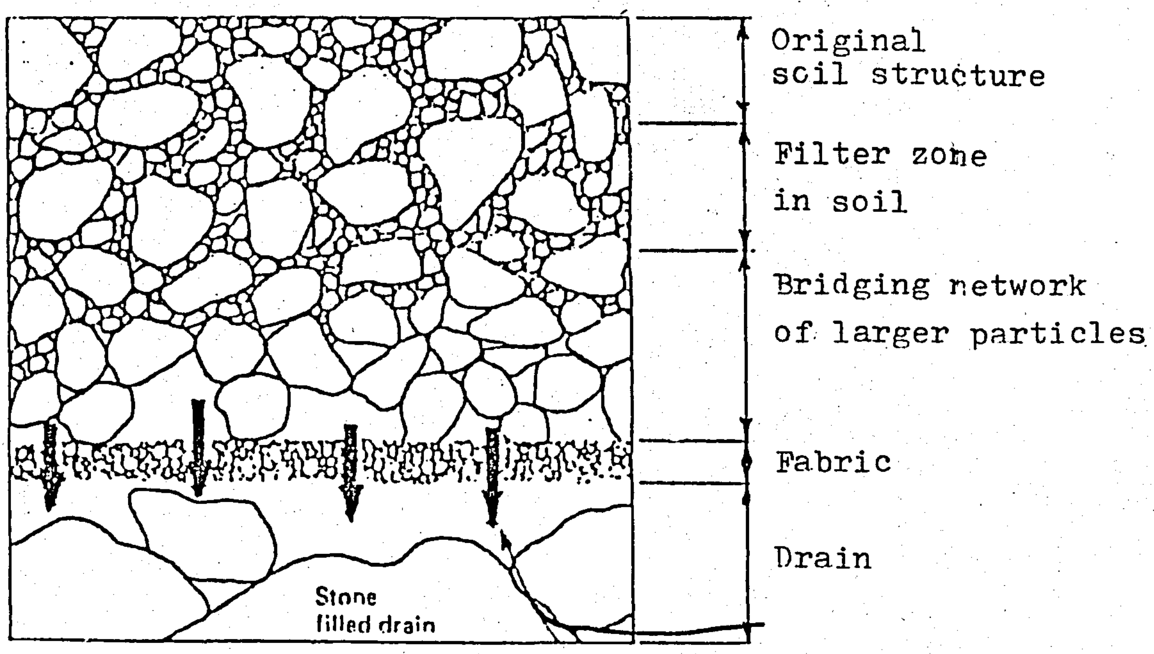


FIG.5.1 EQUILIBRIUM SOIL CONDITIONS FOLLOWING FORMATION OF SOIL FILTER (Hoare,1982)

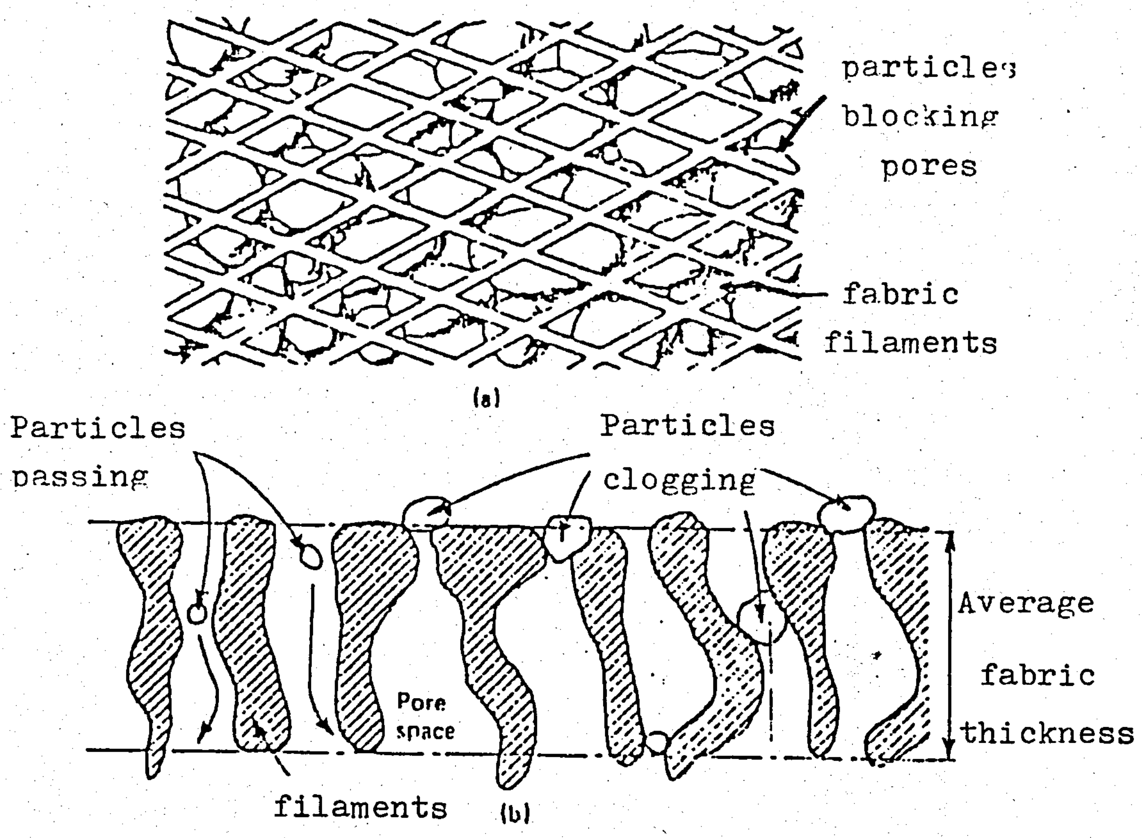


FIG.5.2 INTERACTION OF SOIL PARTICLES AND FABRICS IN DRAINS

a.Particles Blocking Pores.

b.Particles Clogging in Complex Pore System

Figure 5.1 shows a highly permeable zone of larger soil particles forming the bridging network immediately adjacent to the geotextile.The finer soil particles which were present in this zone before installation of the geotextile have all been washed into the drainage system.Immediately behind this bridging zone consisting of soil particles whose permeability decreases as the distance from the geotextile increases.This zone has been termed a "filter cake".The term filter cake has been generally supplanted by the term"soil filter ".The zone behind the soil filter is the extremity of the existing soil which has remained undisturbed throughout the formation of the bridging and soil filter zones.Once the soil filter zone has been established,no further soil is washed through and the system is considered to be in equilibrium.(Lawson,1982).

A closer examination of the structure of the soil filter zone shows it to consist of larger soil particles at the extremity farthest away from the geotextile.This soil filter zone is,in effect,a reverse granular filter constructed solely from the in situ soil particles,and thus will always remain compatible with the undistrubed in situ soil(whereas with conventional granular filters,this is not always the case.)

As soil water continues to flow through the completed soil filter-geotextile system,the soil filter zone actively filters out the soil water from the undisturbed soil mass while the geotextile retains the soil filter zone in place, preventing collapse into drainage layer.Thus,the function of the geotextile in one-directional filter applications is not to filter actively the water from the soil long term but,rather,to act as a type of catalyst in the formation of a stable soil filter from the in situ parent soil.While the geotextile does not actively act as a filter once the soil filter is formed,the choice of the correct geotextile is critical to the formation of a stable effective soil filter.

For ideal filter performance,the permeability of the soil filter,bridging network and geotextile,always should be equal to or greater than the permeability of the in situ soil.Should the permeability of any of these zones fall substantially below that of the undisturbed in situ soil,then reduced water flows into the drainage layer will occur which may result in less than optimal performance from the filter system.

To achieve optimal filter performance,two criteria must be met:

Criterion 1. " Permeability criterion ".Following an initial period of instability which occurs during the formation of the soil filter,the permeability of the system should remain relatively contant with time.(See Fig.5.3 a).

Criterion 2. " Piping Criterion ".Following an initial period of soil piping which occurs during the formation of the soil filter,no further in situ soil should be piped through the filter system.(See Fig.5.3 b).

If the initial criterion is not adhered to,then the permeability of the soil filter zone so formed may continue to decrease and may lead to potentially damaging build-ups in pore pressure behind the filter zone.

If the second criterion is not adhered to,then in situ soil may be continually piped through the filter system which can lead to internal erosionfailures in the structure.

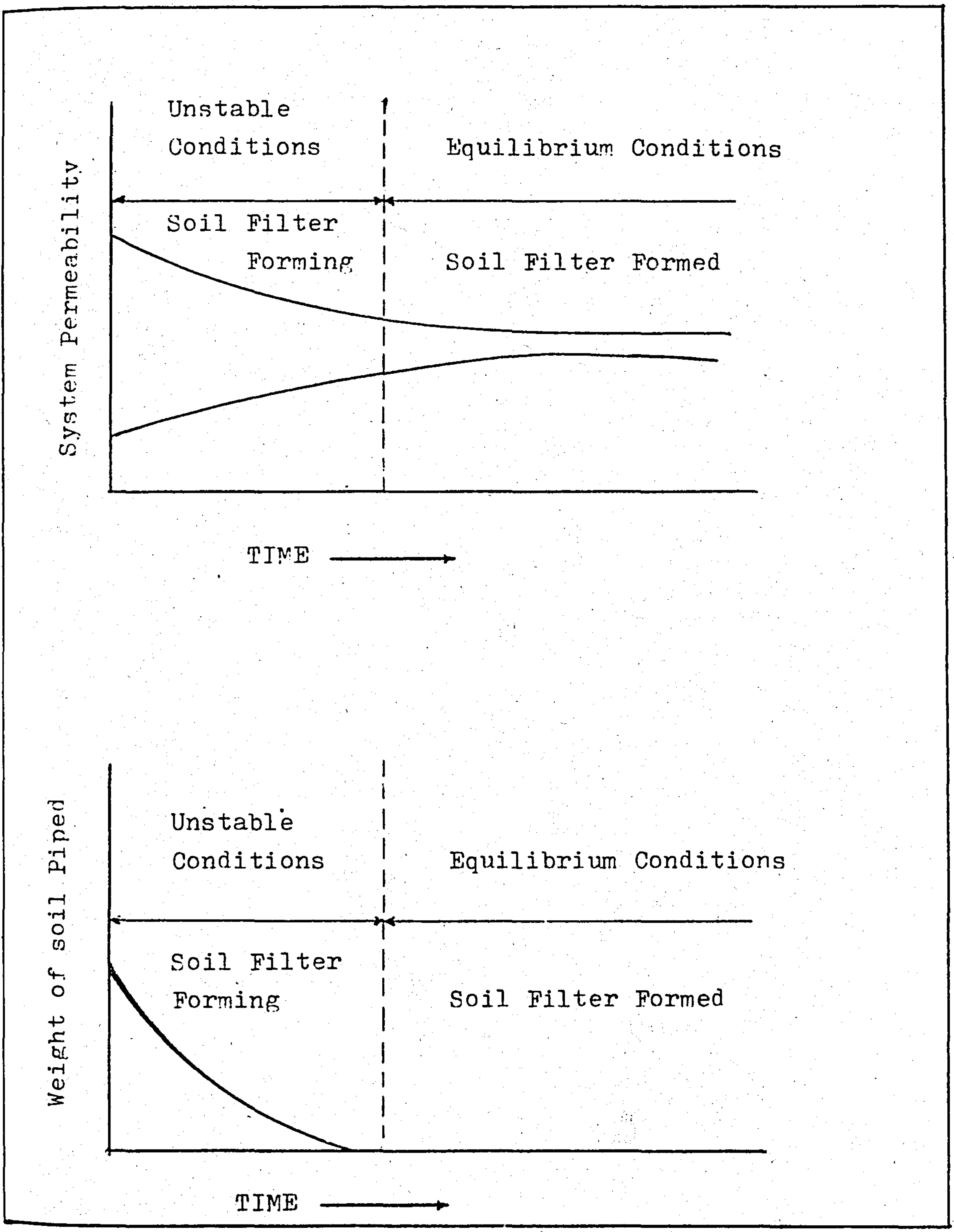


FIG.5.3 GENERAL REQUIREMENTS FOR OPTIMAL FILTER PERFORMANCE

To satisfy both of prceeding criteria, the interaction between the in situ soil and specific properties of the geotextile must be determined adequately. The two major geotextile properties which affect the formation of a stable soil filter are its indicative pore size and its water permeability. Indicative pore size determines the maximum size of soil particle which can migrate across the boundaries of the geotextile. Geotextile permeability determines the number of pores per unit area in the geotextile. For good filtration, it is a requirement that for a given indicative pore size, the optimal geotextile should have as high a permeability as possible, so that when particle blocking of pores during the formation of the bridging network adjacent to the geotextile, it does not reduce critically the permeability of the geotextile. (Lawson, 1982).

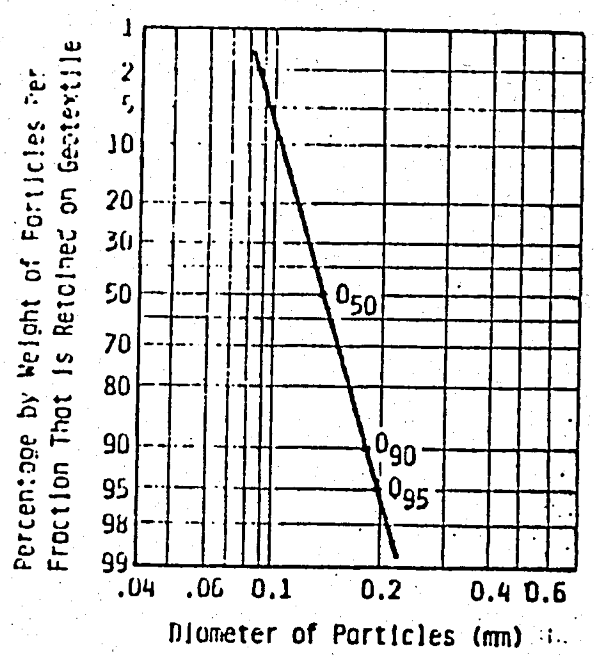


FIG. 5.4 SEMILOGARITHMIC PROBABILITY-NET FOR GEOTEXTILE PORE SIZES (Lawson, 1982).

There are three basic elements of filter criteria for drainage fabrics:

1. Retention Ability(Piping Resistance).

2. Water Permeability.

3. Clogging Resistance.

The characteristics of each criterion are described as follows:

5.2.1- Retention Ability (Piping Resistance)

Retention ability can be specified by using equations given by different authors. Some of them are listed below:

Carroll(1983) defines the retention ability as:

$$\frac{EOS_{fabric}}{D_{85\ soil}} < 2 \text{ or } 3 \qquad \ldots(5.1)$$

where,

EOS_{fabric} = equivalent opening size of the fabric.

$D_{85\ soil}$ = soil diameter below which lie 85% of soil particles.

Equivalent Opening Size:

150 gr. of single size sand is sieved for 20 minute over a fabric on a sieve using an automatic sieve-shaker. The "Equivalent Opening Size" (EOS) is the retained, on the size of that sand fraction of which 5% of the sand by weight passes the fabric.(Hoare,1982).

Calhoun(1972) gives the retention ability of a geotextile as:

$$\frac{P_{95}\text{ fabric}}{D_{85}\text{ soil}} \leqslant 1 \qquad \ldots(5.2)$$

where,

P_{95} fabric = 95% pore size of filter fabric.(See Fig 5.4)

D_{85} soil = soil diameter below which lie 85% of soil particles.

Rankilor(1978) gives the retention ability of a geotextile as:

$$\frac{P_{50}\text{ fabric}}{D_{85}\text{ soil}} \leqslant 1 \qquad \ldots(5.3)$$

where,

P_{50} fabric = 50% pore size of filter fabric.(See FIG 5.4)

D_{85} soil = soil diameter below which lie 85% of soil particles.

Cedergren(1977) gives the retention ability of a geotextile as:

$$\frac{P_{85}\text{ fabric}}{D_{85}\text{ soil}} \leqslant 1 \qquad \ldots(5.4)$$

where,

P_{85} fabric = 85% pore size of filter fabric.(See Fig.5.4)

D_{85} soil = soil diameter below which lie 85% of soil diameter.

Ogink(1975) gives the retention ability of a geotextile as:

-For woven geotextiles: $$\frac{P_{50}\ \text{fabric}}{D_{90}\ \text{soil}} \leq 1 \qquad \ldots(5.5)$$

-For nonwoven geotextiles: $$\frac{P_{50}\ \text{fabric}}{D_{90}\ \text{soil}} \leq 1.8 \qquad \ldots(5.6)$$

where,

P_{50} fabric = 50% pore size of filter fabric(See Fig.5.4)

D_{90} soil = soil diameter below which lie 90% of soil particles.

Schober and Teindl(1979) give the retention ability of a geotextile as:

$$\frac{P_{90}\ \text{fabric}}{D_{50}\ \text{soil}} = B \qquad \ldots(5.7)$$

where,

P_{90} fabric = 90% pore size of filter fabric.(See Fig.5.4)

D_{50} soil = soil diameter below which lie 50% of soil particles.

B = a function of the uniformity coefficient of the soil to be filtered.(See Fig.5.5)

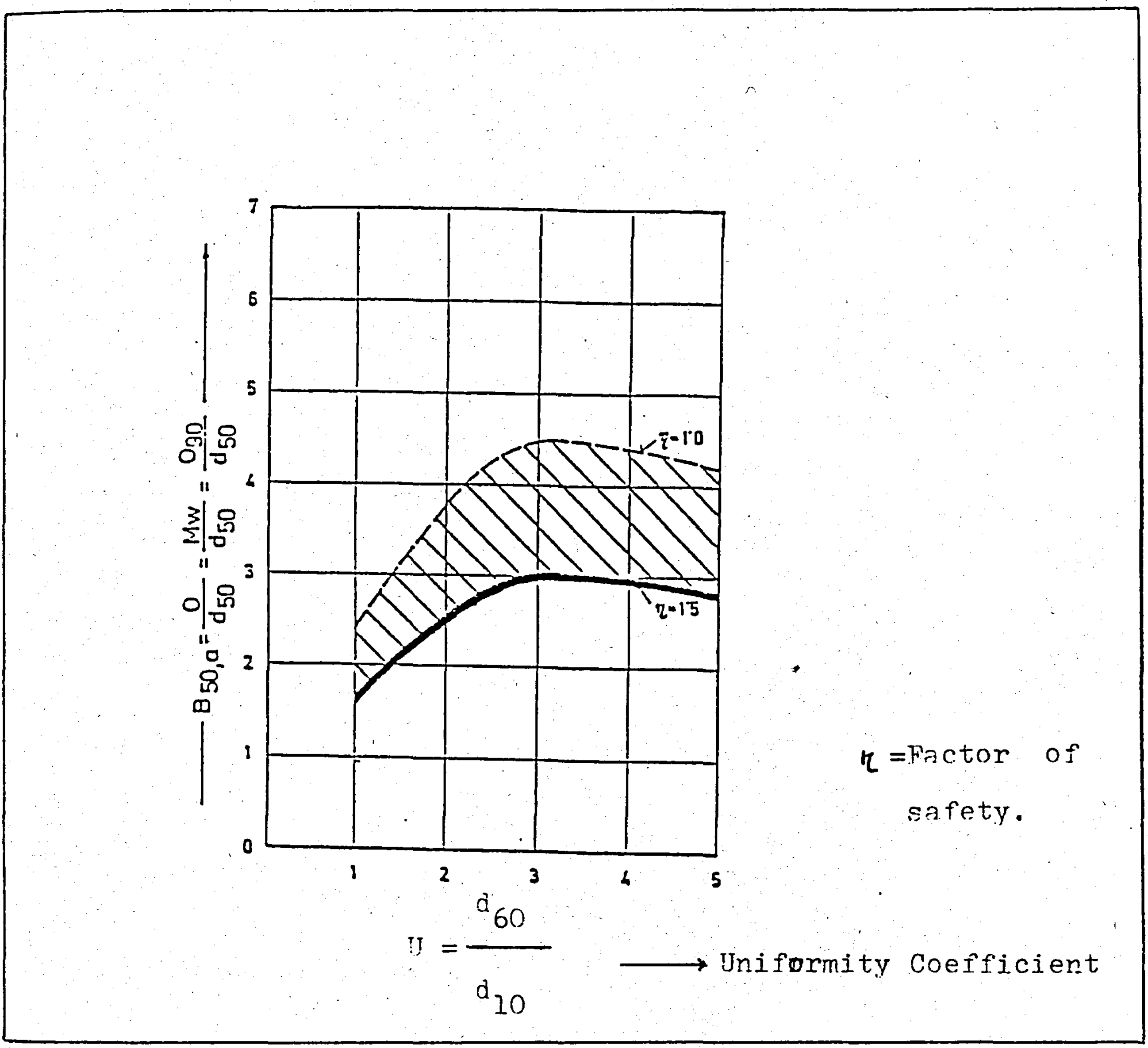


FIG.5.5 FILTER - CRITERION FOR WOVEN AND NONWOVEN GEOTEXTILES(Thickness smaller than 1 mm) WITH STEADY LAMINAR FLOW (Schober and Teindl,1979).

5.2.2 Permeability Requirement

The permeability of a fabric filter should be substantially greater than that of the protected soil in order that, if partial clogging should occur, the fabric permeability will not be reduced a critical level, i.e., below that of protected soil. Accordingly the fabric permeability should be less than that of the protected soil.

Calhoun (1972) gives the requirement to satisfy the permeability condition as:

$$\frac{k_{fabric}}{k_{soil}} \geq 10 \qquad \ldots(5.8)$$

where,

k_{fabric} = coefficient of permeability of the fabric.

k_{soil} = coefficient of permeability of the protected soil.

Marks(1975) gives the requirement to satisfy the permeability condition as:

$$\frac{k_{fabric}}{k_{soil}} \geq 5 \qquad \ldots(5.9)$$

where,

k_{fabric} = coefficient of permeability of the fabric.

k_{soil} = coefficient of permeability of the protected soil.

Rankilor (1978) gives the requirement to satisfy the permeability condition as:

$$\frac{P_{50\ \text{fabric}}}{D_{15\ \text{soil}}} \geq 1 \qquad \ldots(5.10)$$

where,

$P_{50\ \text{fabric}}$ = 50% pore size of the filter fabric.

$D_{15\ \text{soil}}$ = soil diameter below which lie 15% of soil particles.

5.2.3 Clogging Resistance

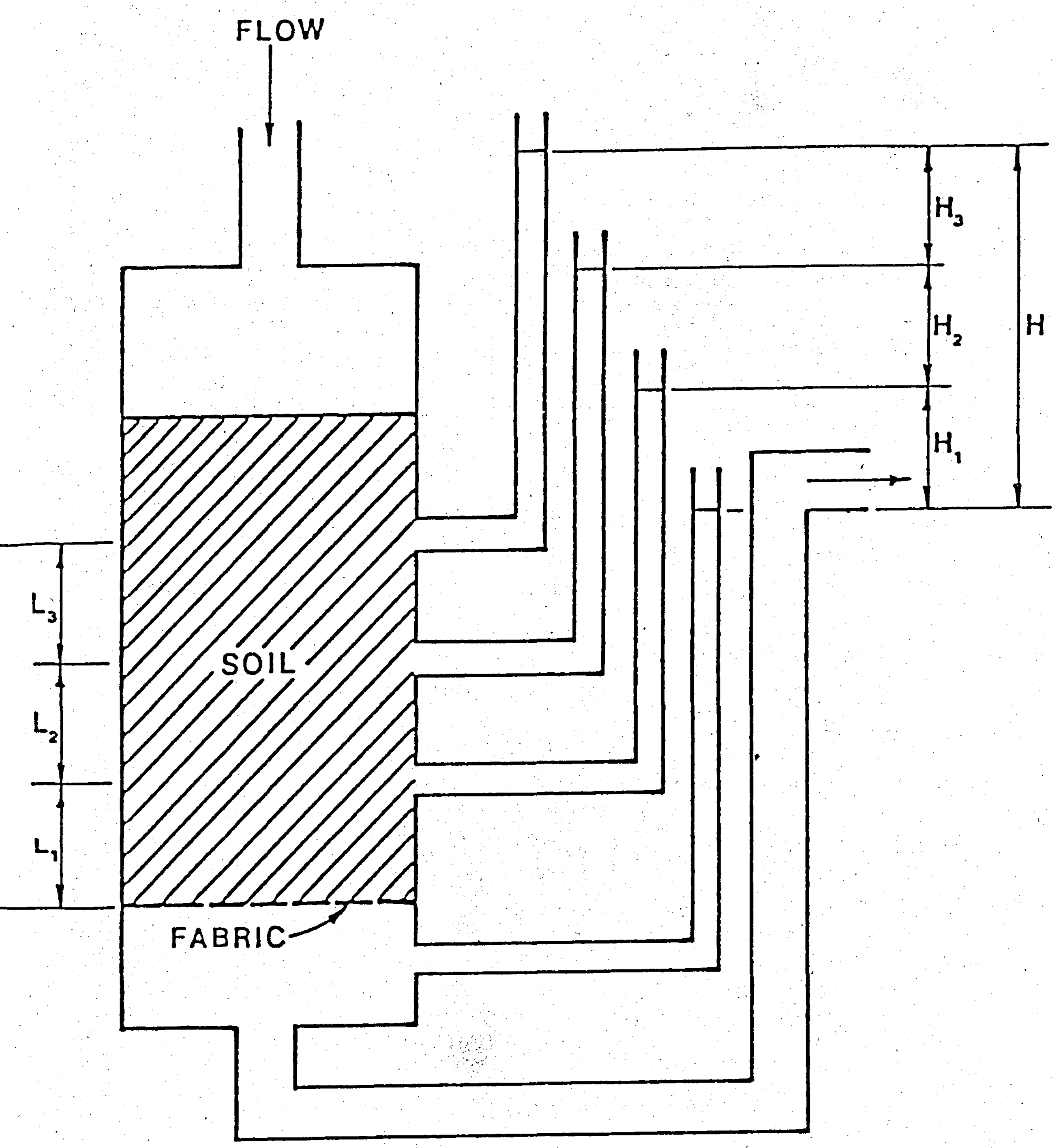
Carroll (1983) relates the clogging resistance to the gradient ratio which is defined in Fig.5.6. The maximum allowable gradient ratio for acceptable filter performance is 3. Therefore, the criterion for clogging resistance of fabric filters can be stated as follows:

$$GR \leq 3 \qquad \ldots(5.11)$$

where,

GR = Gradient Ratio of the Filter Fabric.

The clogging behaviour of a geotextile should be evaluated in a test that simulates in-use conditions.



$L_1 = L_2 = L_3 = 1.0$ IN.

$$\text{GRADIENT RATIO} = \frac{i_1}{i_{2+3}} = \frac{\dfrac{H_1}{L_1}}{\dfrac{(H_2 + H_3)}{(L_2 + L_3)}}$$

FIG. 5.6 ILLUSTRATION OF PROCEDURE FOR COMPUTING U. S. ARMY engineer GRADIENT RATIO (Carroll , 1983)

5.3- ADVANTAGES AND DISADVANTAGES OF FABRIC FILTERS

In addition to the advantage of inharent tensile strength , fabric filters have several other advantages over granular filters.In general ,installation should be quicker and more labor-efficient , and the local availability of suitable granular filter material is no longer a design consideration.Since a fabric's filtering ability is factory-controlled , it cannot be altered by careless placement by site labor , and a quick visual inspection assures the engineer that it is in place as designed.

Potential disadvantages are that installation must be undertaken with due care so as to prevent undue exposure to ultraviolet light and so that the fabric does not become torn or damaged with adequate overlaps between sheets provided.The life of a fabric in a soil environment is also as yet known and unproved over the lifetime of a normal engineering structure.

CHAPTER 6

EXPERIMENTAL STUDY

6.1- THE PURPOSE OF THE TESTS

The purpose of the tests is to determine the coefficient of permeability of (Saturated Clay + Geotextile + Wet Gravel Filter)system,and the variation with time.For this reason, firstly,the coefficient of permeability of clay is determined. The coefficient of permeability of the geotextile is known. And then,the coefficient of permeability of (Saturated Clay + Geotextile + Wet Gravel Filter) system is determined. The coefficient of permeability of clay and the coefficient of permeability of (Saturated Clay + Geotextile + Wet Gravel Filter) system are compared with each other,and the difference is determined.If there is a difference,one may say,there must be clogging in the pores of the geotextile.

6.2- TEST APPARATUS

Certain modification to CBR (California Bearing Ratio) test molds have been carried out to develop a special permeability device,which consists of a cylindirical wall and two friction-fitted covers(the bottom one being conical as shown in Fig. 6.1.

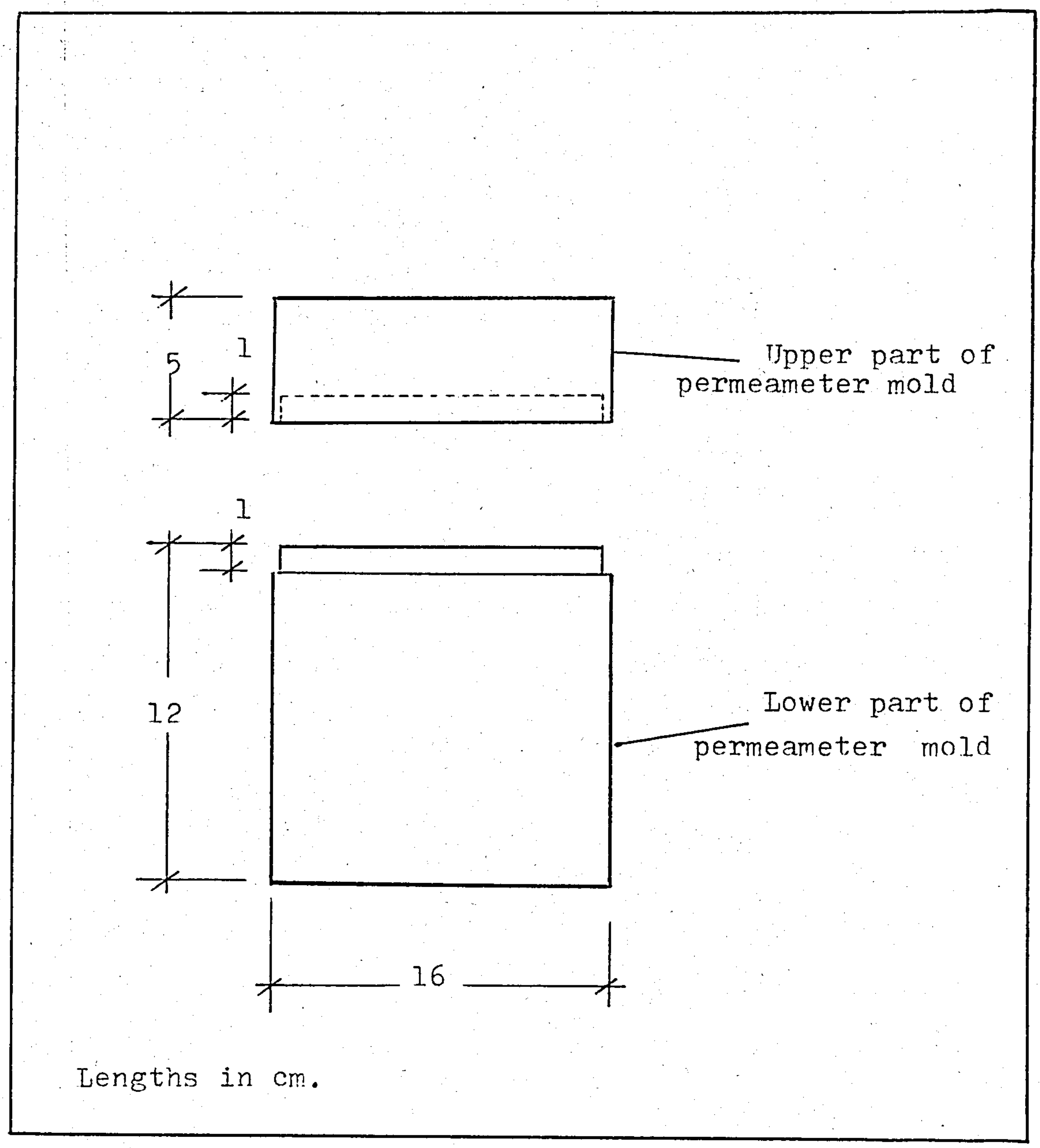


FIG. 6.1 SCHEMATIC OF THE PERMEAMETER MOLD

This used steel molds are made of two parts(height of the first one is 5 cm.,and height of the second one is 12 cm.),and both upper and bottom parts have diameters of 7.62cm.(3 in.).

This permeability device is connected to a 50 ml.burette using a plastic tube.The connections and edges of mold which are covered by upper and bottom cover are sealed throughly to make waterproof using overrings and silicone (which is a kind of mastic).There is a valve at the bottom of the mold to collect water passing through the test sample.This device is attached to a standard laboratory ring stand using a test-tube clamp, A meterstick is used to obtain initial head h_1,and final head h_2.The meterstick is also attached to ring stand using a test -tube clamp.Schematic of the permeameter setup is shown in Fig. 6.2.

6.3- LIMITATIONS OF THE TESTS

1.The soil in the permeability device is never in the same state as in the field.It is always disturbed to some extent.

2.Boundary conditions are not the same in the laboratory. The smooth walls of the permeability mold make for better flow paths than if they were rough.If the soil is stratified vertically ,the flow in different strata will be different,and this boundary condition may be impossible to duplicate.

3.The hydraulic head h may be different(often much larger) in the laboratory,causing a washout of fine material

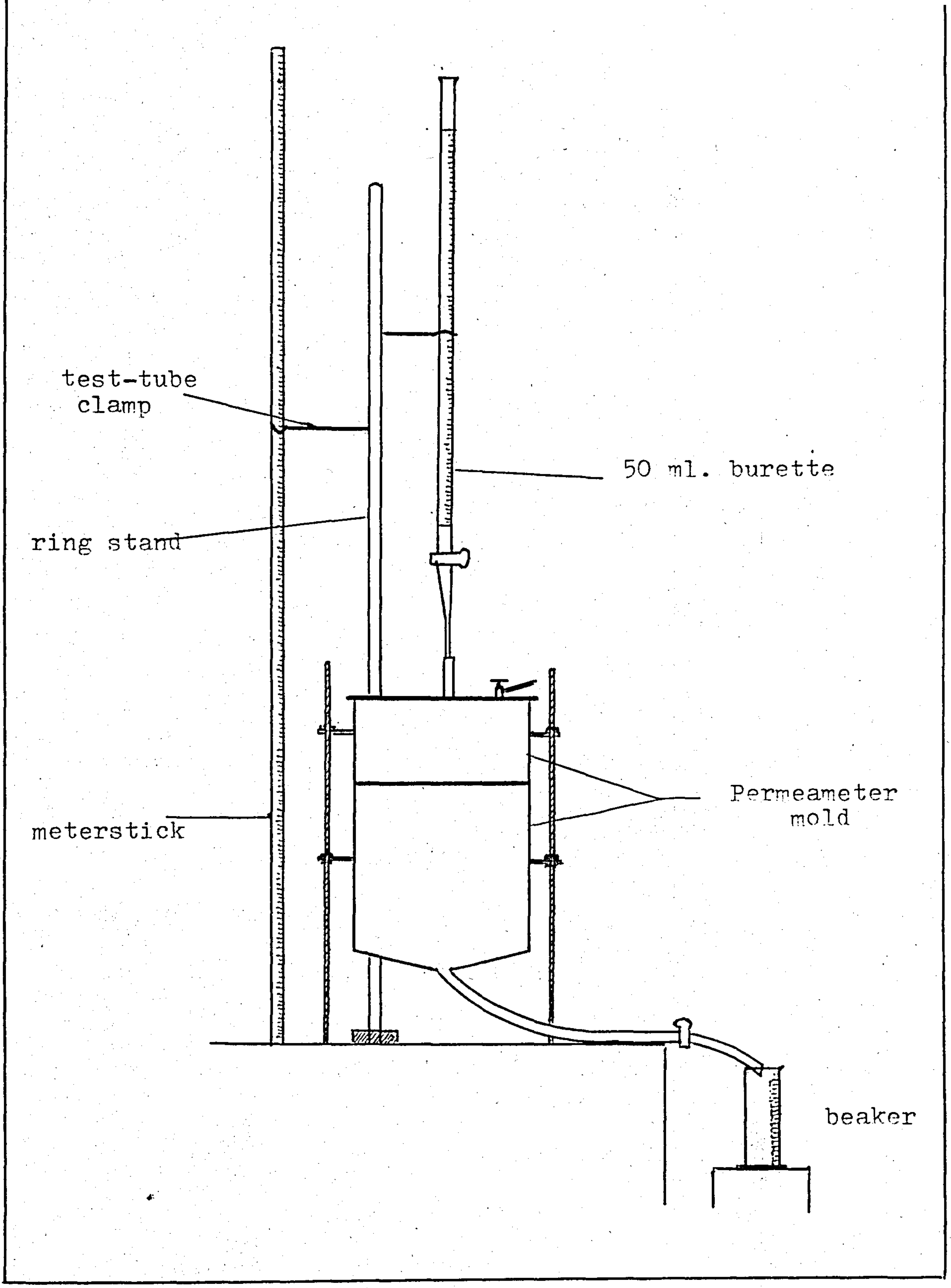


FIG. 6.2 SCHEMATIC OF THE PERMEAMETER SETUP

to the boundary,with a possible reduction of k.The field hydraulic gradient(i= h/L) is on the order of 0.5 to 1.5, whereas in the laboratory it is generally 5 or more. Some evidence indicates that $V = ki$ is not linear for all values of i,especially the larger values.On the other hand,there is evidence that for fine-grained soils(clays),there may be some threshold gradient below which no flow will take place.

4.The effect of entrapped air on the laboratory sample will be large even for small air bubbles since the sample is small.

6.4- TEST PROCEDURE

1.The clayey soil is saturated in standard compaction test mold,placing this in a sink in which water is about 5 cm.(2 in.) above the cover.The outlet pipe must be open so that water can back up through the sample.This procedure will saturate the sample with a minimum of entrapped air.When water in the plastic inlet tube on top of the mold reaches equilibrium with the water in the sink (allowing for capillary rise in the tube), the sample may be assumed to be saturated. A soaking period of 24 hr. might provide better results.On the other hand,the wet gravel is placed in the bottom part of the mold,and between the upper and bottom mold geotextile is placed.A piece of filter paper is placed on bottom of the gravel.

2.When water level in the plastic inlet tube is stabilized ,the permeameter is removed from the sink clamping the exit tube. The saturated clay in this standard permeameter is poured into the upper part of the test mold. And then, the upper cover is closed and the rim of the mold are sealed throughly and made waterproof using silicone. The inlet tube of mold is attached to burette, which has been fastened to a ring stand. When the silicone is dried, the test may be started.

3. The lines at the top of the clay are deaired by opening the hose clamp from the burette and opening the petcock on top of the mold. Waetr is allowed to flow(but keep adding water to the burette so it does not become empty) from the petcock until air bubbles cease to exist; then the petcock is closed. The inlet tube from the burette is not closed. The exit tube is still clamped shut.

4. The burette is filled to a convenient height, and, the hydraulic head across the sample is measured to obtain initial head h_1.

5. The exit tube is opened and simultaneously the timer is started. Water is allowed to flow through the sample until the burette is almost empty. Simultaneously the elapsed time is recorded and only the exit tube is clamped. The hydraulic head across the sample at this time is measured to obtain final head h_2. The temperature of the test is recorded.

6. The burette is refilled and Step 5 is repeated three additional times. The temperatures of each run are recorded.

7. Using the Eqn. 6.1 given below, the coefficient of the permeability of the system is computed:

$$k_T = \frac{2.3\ a\ L}{A\ t} \log \frac{h_1}{h_2} \qquad \ldots(6.1)$$

where,

k_T = coefficient of permeability at test temperature T.

a = cross-sectional area of the burette, in cm^2.

L =length of soil sample,in cm.

t = elapsed time of test,in sec.

h_1 = hydraulic head across sample at the beginning of the test (t=0) , in cm.

h_2 = hydraulic head across sample at the end of the test ,in cm.

From Table 3.1 viscosity corrections for μ_T / μ_{20} are found and multiplying these by k_T, the coefficients of permeability at standard temperature (20^oC) may be computed.

$$k_{20} = k_T \frac{\mu_T}{\mu_{20}} \qquad \ldots(6.2)$$

8.After finishing these tests,the apparatus is reassembled and geotextile is left to dry at laboratory temperature along two days ,and then all steps are repeated.

6.5- PROPERTIES OF USED SOILS

6.5.1.Properties of Esan Yellow Clay:

Some laboratory tests have been performed to determine the some soil characteristics of clay. These tests and their results are given below:

.Sieve Analysis: See Fig. 6.3.

.Specific Gravity Test:

Specific Gravity of Esan Yellow Clay, $G_s = 2.72$

.Compaction Test:

Maximum Dry Density Of Clay, max $\gamma_{dry} = 15.30$ kN/m^3.

Optimum Moisture Of Clay, $w_{opt} = 24\%$

.Atterberg Limits Test:

Liquid Limit of Clay, $w_L = 37\%$

Plastic Limit of Clay, $w_P = 20\%$

Plasticity Index of Clay, $I_p = 17\%$

.Saturation Degree Test:

Saturation Degree of Clay, $w_{sat} = 54\%$

.Permeability Test:

The coefficient of Permeability of Esan Yellow Clay , $k_{20} = 1.241 \times 10^{-5}$ cm/sec.

6.5.2.Properties of Brown Clay:

.Sieve Analysis: See Fig.6.4.

.Specific Gravity Test:

Specific Gravity of Brown Clay, $G_s = 2.76$.

.Compaction Test:

Maximum Dry density of Clay, max $\gamma_{dry} = 12.1$ kN/m^3.

Optimum Moisture Content of Clay, $w_{opt} = 33.8$ %

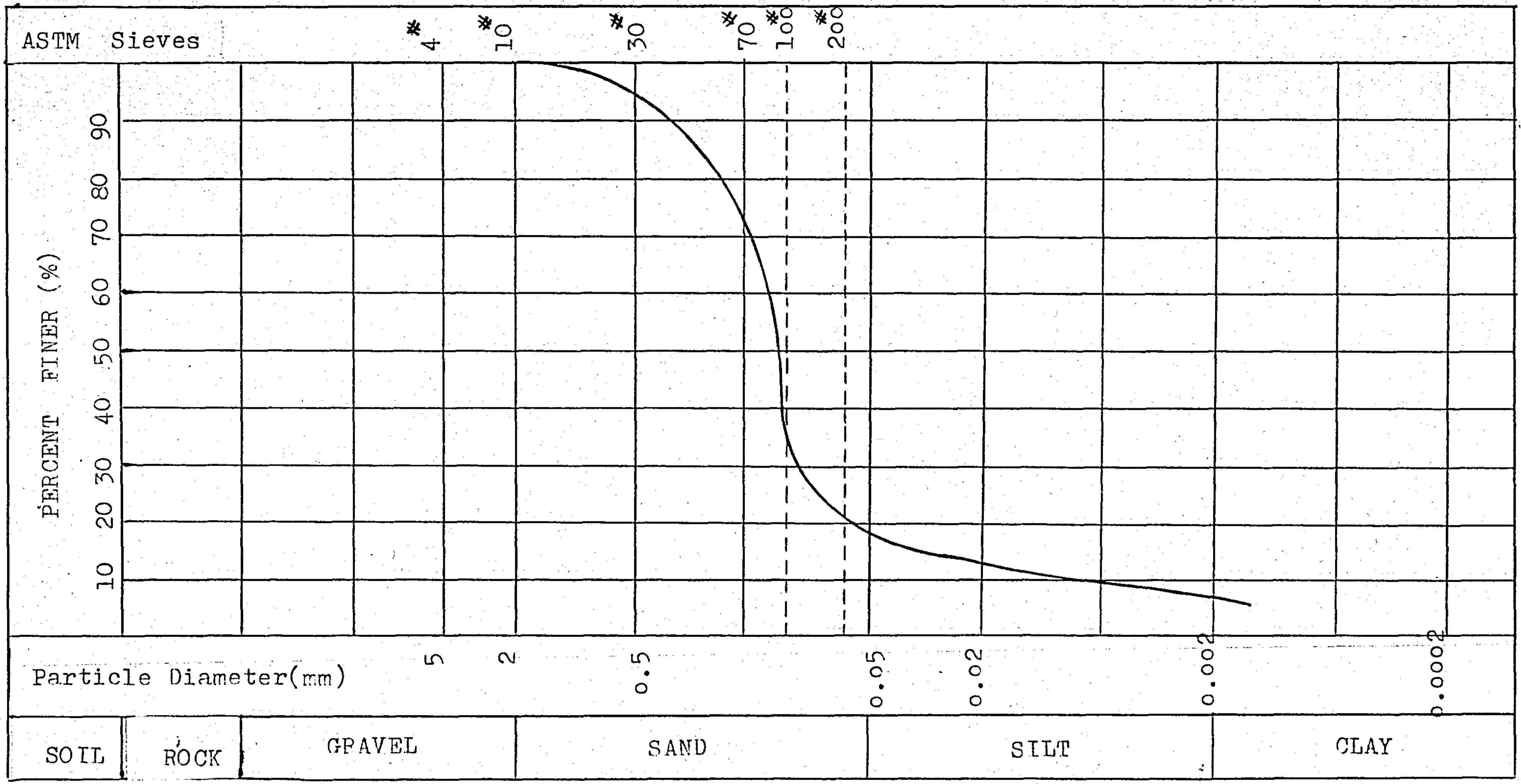


FIG. 6.3. GRANULOMETRIC CURVE FOR ESAN YELLOW CLAY

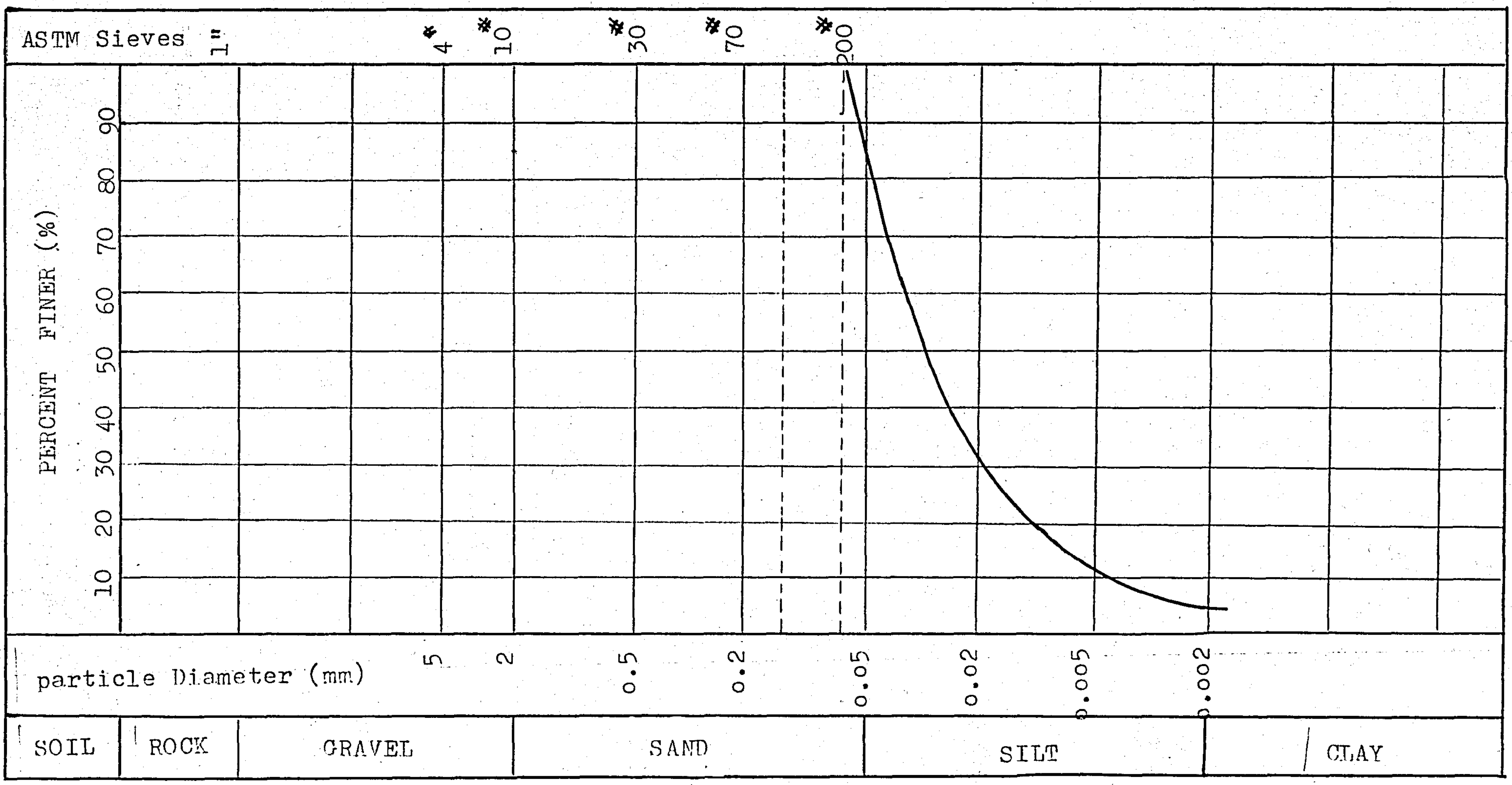


FIG. 6.4. GRANULOMETRIC CURVE FOR BROWN CLAY

.Atterberg Limits Test:

Liquid Limit of Clay, $w_L = 73\%$

Plastic Limit of Clay, $w_p = 32.5\%$

Plasticity Index of Clay, $I_p = 40.5\%$

.Saturation Degree Test:

Saturation Degree of Clay, $w_{sat} = 98\%$

Permeability Test:

The coefficient of Permeability of Brown Clay,

$k_{20} = 5.21 \times 10^{-6}$ cm/sec.

6.5.3. Properties of Gravel:

Sieve Analysis: See Fig.6.5.

6.6- PROPERTIES OF USED GEOTEXTILES

Four types of geotextile have been used in tests. These are TYPAR 3207 , TYPAR 3407-2 , TYPAR 3807-4 , and MIRAFI P40. Properties of these geotextiles are given below:

6.6.1. Properties of TYPAR 3207:

Nonwoven Polypropylene Geotextile

.Unit Weight = 68 gr/m^2.

.Thickness at 2 kN/m^2 = 0.36 mm.

.K value at 2 kN/m^2 = 28 x 10^{-4} m/sec.

.Flow at 1 cm. water head = 45 l/m^2.sec.

.Flow at 10 cm.water head =200 l/m^2.sec.

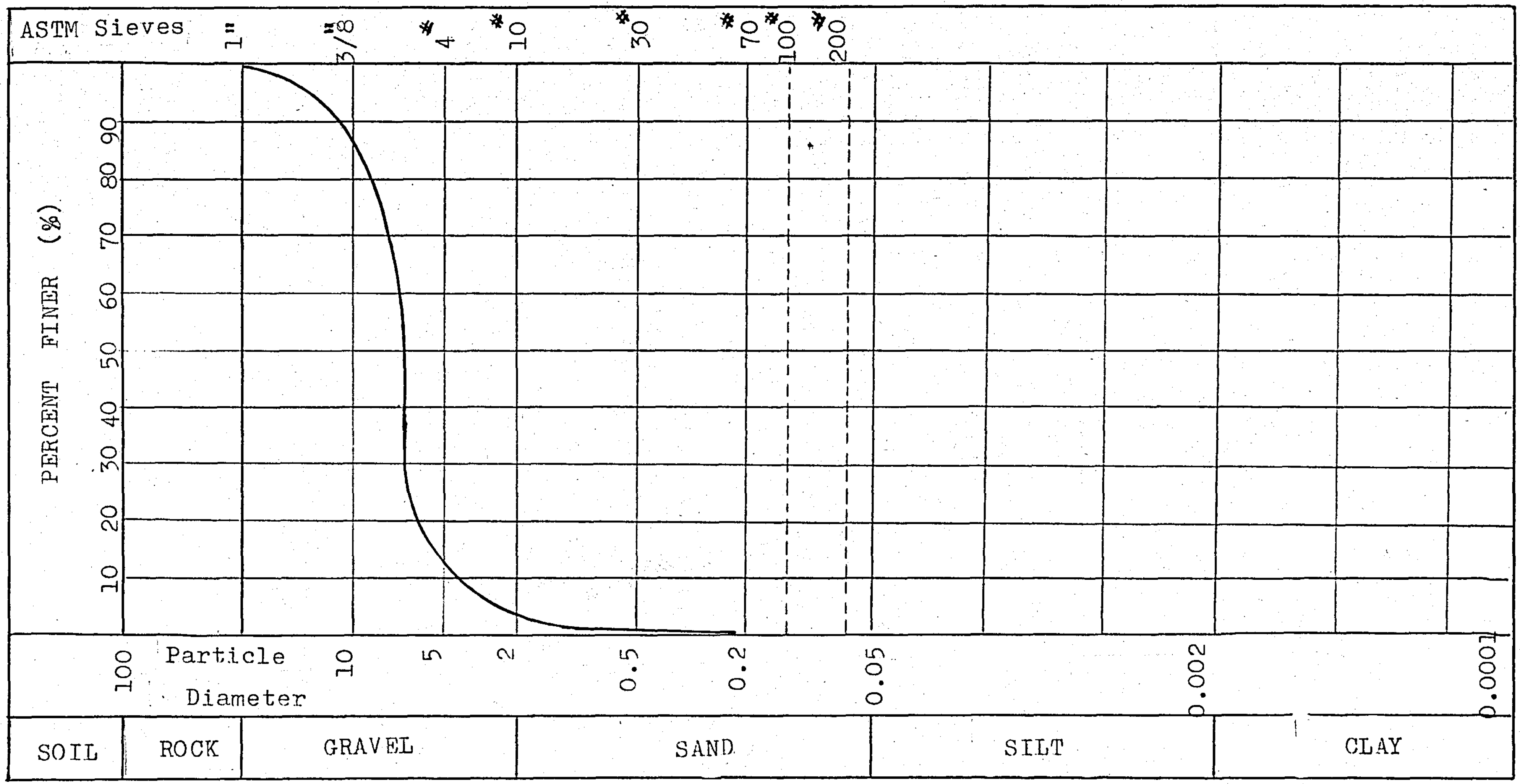


FIG. 6.5 GRANULOMETRIC CURVE FOR GRAVEL

.Sieve Analysis, P_{50} = 350 μm.

P_{90} = 435 μm.

P_{98} = 490 μm.

.Dynamic Filtration, P_{95} = 126 μm.

6.6.2. Properties of TYPAR 3407-2:

Nonwoven Polypropylene Geotextile

.Unit weight = 150 gr/m^2.

.Thickness at 2 kN/m^2 = 0.49 mm.

.K value at 2 kN/m^2 = 5.1 x 10^{-4} m/sec.

.Flow at 1 cm. water head = 11 l/m^2.sec.

.Flow at 10 cm.Water head = 78 l/m^2.sec.

.Sieve Analysis, P_{50} = 90 μm.

P_{90} =130 μm.

P_{98} =160 μm.

.Dynamic Filtration, P_{95} = 108 μm.

6.6.3. Properties of TYPAR 3807-4.

Nonwoven Polypropylene Geotextile

.Unit Weight = 280 gr/m^2.

.Thickness at 2 kN/m^2 = 0.71 mm.

.K value at 2 kN/m^2 = 1.6 x 10^{-4} m/sec.

.Flow at 1 cm. water head = 2.5 l/m^2.sec.

.Flow at 10cm. water head = 20 l/m^2.sec.

.Sieve Analysis, P_{50} = 40 μm.

P_{90} = 60 μm.

P_{98} = 85 μm.

.Dynamic Filtration, P_{95} = 40 um.

6.6.4 Properties of MIRAFI P40.

Nonwoven Polyester Geotextile

- Unit Weight = 150 gr/m^2.
- Thickness = 2.3 mm.
- Specific Gravity = 1.38
- Opening Size = 150 µm.
- The coefficient of permeability, $K = 2.0 \times 10^{-1}$ cm/sec.

6.7- TEST RESULTS

6.7.1. For (SATURATED ESAN YELLOW CLAY + TYPAR 3207 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used standpipe, a = 0.988 cm^2.

Length of (Saturated Clay + Geotextile) layer, L = 5 cm.

...Using virgin geotextile ....

...1^{st} TEST.......

TEST NO	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	1114	25	25	18
2	80.6	55.3	1246	25	25	17.5
3	80.6	55.3	1403	25	25	17.5
4	80.6	55.3	1541	25	25	17
Average	80.6	55.3	1326	25	25	17.5

$$k_{ave} = \frac{2.3\ (0.988)\ 5}{(182.415)\ 1326} \log \frac{80.6}{55.3}$$

$$k_{ave} = 8.177 \times 10^{-6} \text{ cm/sec.}$$

6.7.2. For (SATURATED ESAN YELLOW CLAY + TYPAR 3207 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used standpipe, a = 0.988 cm^2.

Length of (Saturated Clay + Geotextile) layer, L = 5 cm.

...Using geotextile which is dried along two days ..... after 1st Test........

...2nd TEST using TYPAR 3207.......

test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	1714	25	25	18
2	80.6	55.3	1943	25	25	17.5
3	80.6	55.3	2018	25	25	17.5
4	80.6	55.3	2154	25	25	18
Average	80.6	55.3	1957	25	25	17.75

$$k_{ave} = \frac{2.3\ (0.988)\ 5}{182.415\ (1957)} \log \frac{80.6}{55.3}$$

$$k_{ave} = 5.505 \times 10^{-6} \text{ cm/sec.}$$

6.7.3 For (SATURATED ESAN YELLOW CLAY + TYPAR 3207 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used standpipe, a = 0.988 cm^2.

Length of (Saturated Clay + Geotextile) layer, L = 5 cm.

...Using geotextile which is dried along two days after 2^{nd} test..........

....3^{rd} TEST using TYPAR 3207..........

TEST NO	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	2246	25	25	17
2	80.6	55.3	2281	25	25	17.5
3	80.6	55.3	2357	25	25	17.6
4	80.6	55.3	2441	25	25	17.1
Average	80.6	55.3	2331	25	25	17.3

$k_{ave} = 4.675 \times 10^{-6}$ cm/sec.

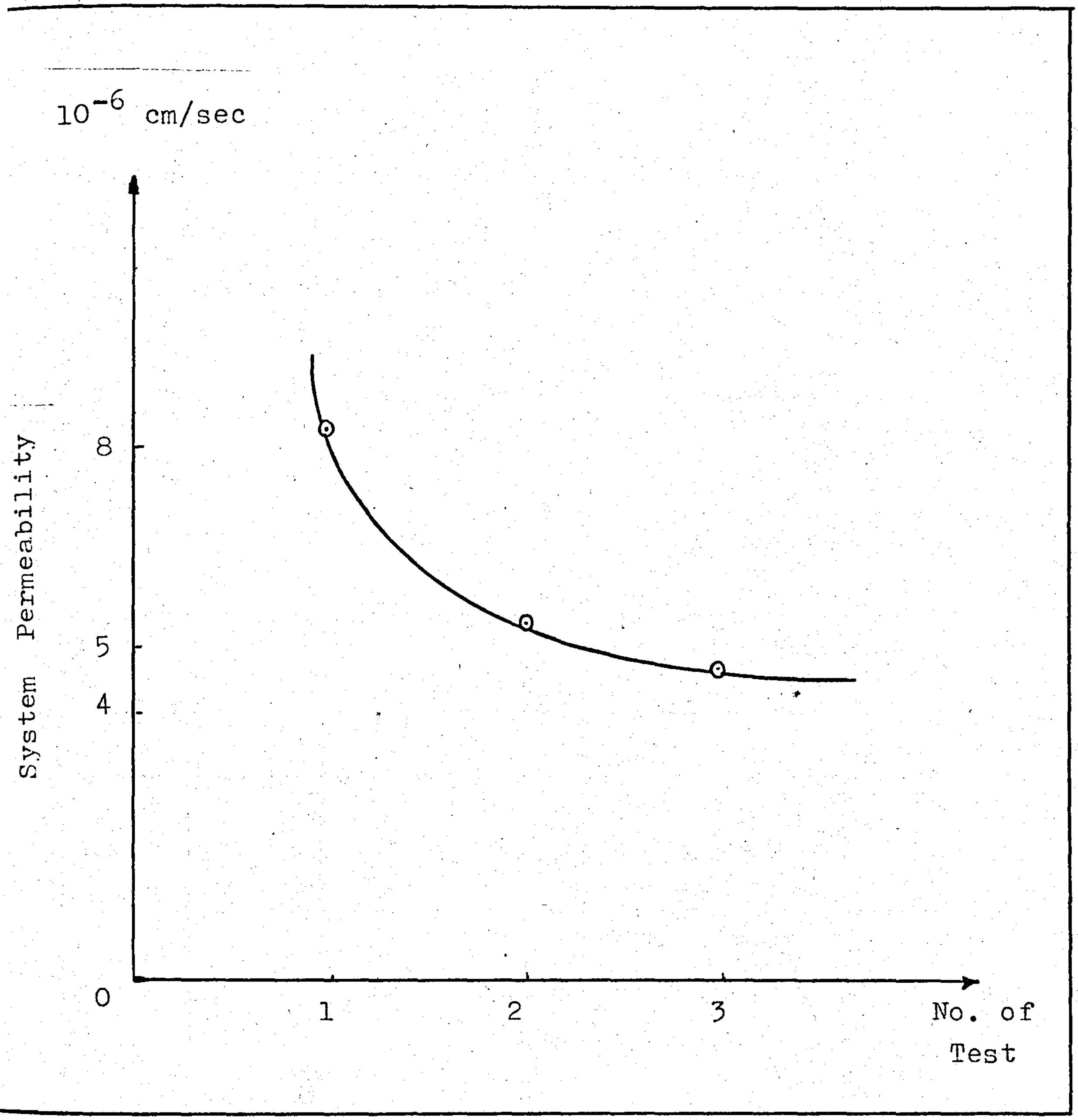


FIG.6.6 NO. of TEST - SYSTEM PERMEABILITY CURVE for (Esan clay + TYPAR 3207 + Wet Gravel Filter) system

6.7.4 For (SATURATED ESAN YELLOW CLAY + TYPAR 3407-2 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used burette, $a=0.988\ cm^2$.

Length of (Saturated Clay + Geotextile) layer, L = 5 cm.

...Using virgin geotextile.......

....1st TEST........

TEST no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	82.9	32.3	6954	50	50	14
2	82.9	32.3	7111	50	50	14.5
3	82.9	32.3	7297	50	50	13.5
4	82.9	32.3	7521	50	50	14
Average	82.9	32.3	7220	50	50	14

$k_{ave} = 4.114 \times 10^{-6}$ cm/sec.

6.7.5. For (SATURATED ESAN YELLOW CLAY + TYPAR 3407-2 + WET GRAVEL FILTER) system.

Used standpipe= 5o ml. burette.

Cross-sectional area of the used burette, a = 0.988 cm^2.

Length of (Saturated Clay + Geotextile)layer, L= 5 cm.

...Using geotextile which is dried along two days after 1st Test.........

...2nd TEST using TYPAR 3407-2.........

TEST no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	82.9	32.3	8165	50	50	15.5
2	82.9	32.3	11988	50	50	16
3	82.9	32.3	16346	50	50	15
4	82.9	32.3	17741	50	50	15
Average	82.9	32.3	13560	50	50	15.4

$k_{ave} = 2.111 \times 10^{-6}$ cm/sec.

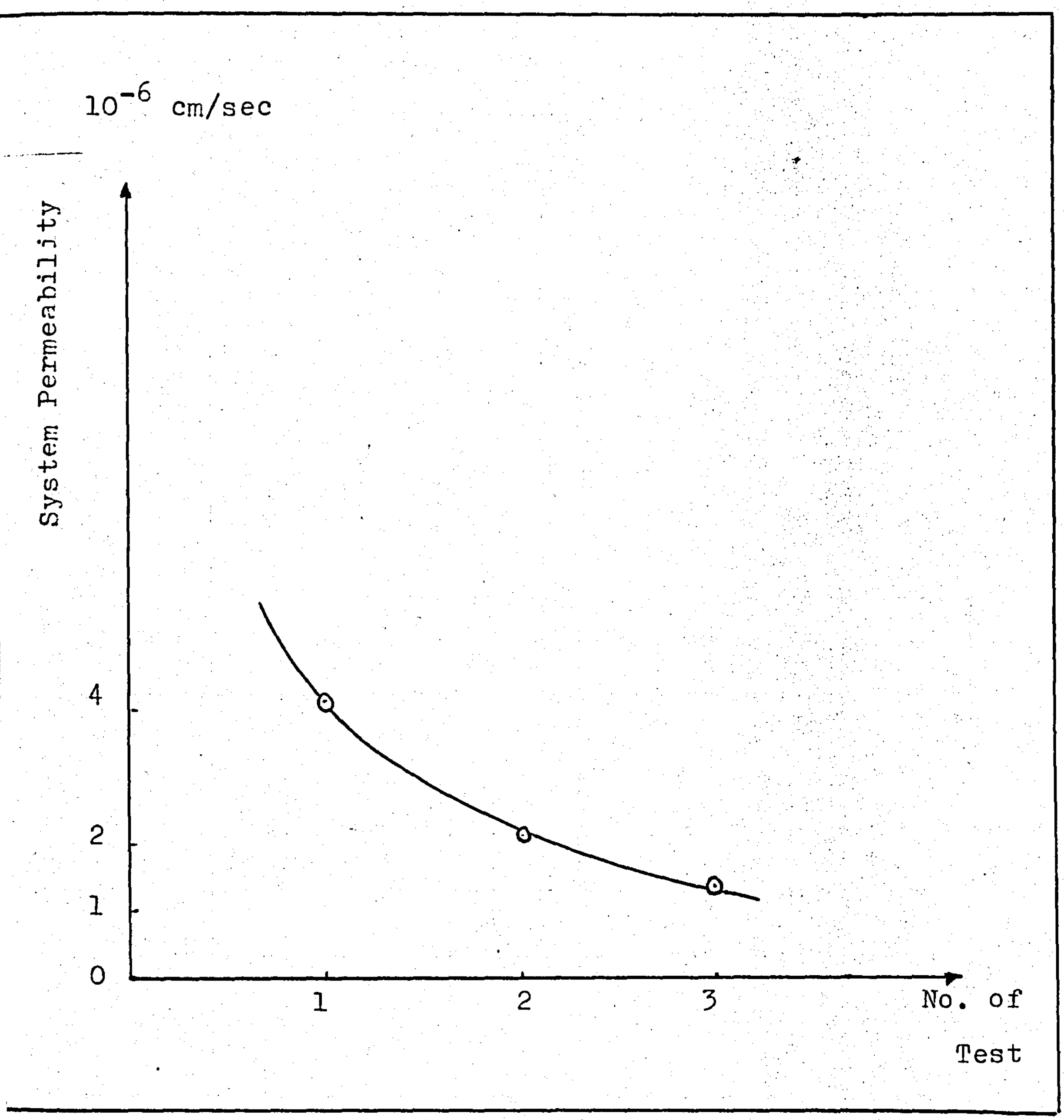


FIG. 6.7 NO. OF TEST - SYSTEM PERMEABILITY CURVE
for (Esan clay + TYPAR 3407-2 + Wet gravel Filter) system

6.7.7 For (SATURATED ESAN YELLOW CLAY + TYPAR 3807-4 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used standpipe, a = 0.988 cm^2.

Length of (Saturated Clay + Geotextile)layer, L = 5 cm.

...Using virgin geotextile.........

...1st TEST using TYPAR 3807-4...........

TEST NO	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.2	29.6	11854	50	50	17.5
2	80.2	29.6	12131	50	50	18.5
3	80.2	29.6	13153	50	50	16.5
4	80.2	29.6	14223	50	50	16.5
Average	80.2	29.6	12840	50	50	17.3

$k_{ave} = 2.25 \times 10^{-6}$ cm/sec.

6.7.8 For (SATURATED ESAN YELLOW CLAY + TYPAR 3807-4 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used burette, $a = 0.988\ cm^2$.

Length of (Saturated Clay + Geotextile) layer, $L = 5$ cm.

...Using geotextile which is dried along two days after 1st Test.........

...2nd TEST using TYPAR 3807-4..........

TEST NO	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.2	29.6	18281	50	50	18
2	80.2	29.6	19543	50	50	19
3	80.2	29.6	20117	50	50	18.5
4	80.2	29.6	21113	50	50	18.5
average	80.2	29.6	19764	50	50	18.5

$k_{ave} = 1.415 \times 10^{-6}$ cm/sec.

6.7.9 For (SATURATED ESAN YELLOW CLAY + TYPAR 3807-4 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used standpipe, $a = 0.988\ cm^2$.

Length of (Saturated Clay + Geotextile) layer, $L = 5$ cm.

...Using geotextile which is dried along two days after 2nd Test.........

...3rd TEST using TYPAR 3807-4............

TEST no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.2	29.6	31957	50	50	15.5
2	80.2	29.6	26842	50	50	15.8
3	80.2	29.6	24011	50	50	16.2
4	80.2	29.6	23894	50	50	19
Average	80.2	29.6	26676	50	50	16.6

$k_{ave} = 1.1 \times 10^{-6}$ cm/sec.

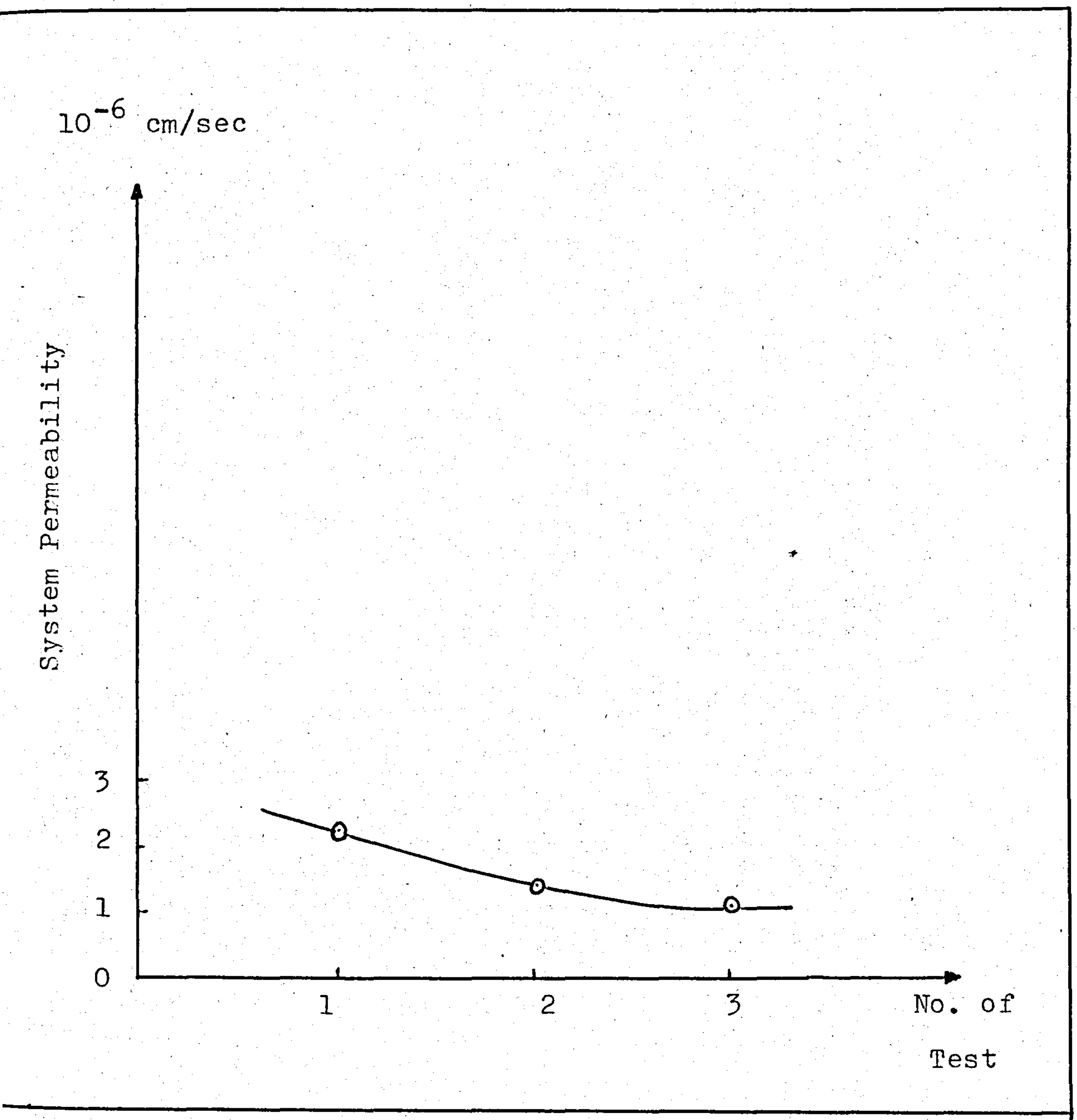


FIG. 6.8 NO. OF TEST- SYSTEM PERMEABILITY CURVE
for (Esan clay + TYPAR 3807-4 + Wet gravel Filter)system

6.7.10. For (SATURATED ESAN YELLOW CLAY + MIRAFI P 40 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used burette, a= o.988 cm^2.

Length of (Saturated clay + geotextile)layer, L= 5 cm.

...Using Virgin Geotextile......

...1st TEST using MIRAFI P40........

TEST no.	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	2341	25	25	17
2	80.6	55.3	2419	25	25	17.5
3	80.6	55.3	2617	25	25	17.5
4	80.6	55.3	2746	25	25	18
Average	80.6	55.3	2531	25	25	17.5

k_{ave} = 4.284 x 10^{-6} cm/sec.

6.7.11. For (SATURATED ESAN CLAY + MIRAFI P 40 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used standpipe, a= o.988 cm^2.

...Using geotextile which is dried along two days after 1st Test........

...2nd TEST using MIRAFI P40...........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	3516	25	25	18
2	80.6	55.3	4130	25	25	18
3	80.6	55.3	4247	25	25	17.5
4	80.6	55.3	4518	25	25	17
Average	80.6	55.3	4102	25	25	17.6

$k_{ave} = 2.636 \times 10^{-6}$ cm/sec.

6.7.12. For (SATURATED ESAN YELLOW CLAY + MIRAFI P 40 + WET GRAVEL FILTER)SYSTEM.

Used standpipe = 50 ml. burette

Cross-sectional area of the used burette, $a = 0.988\ cm^2$.

Length of(saturated clay + geotextile) layer, $L \triangleq 5$ cm.

...Using geotextile which is dried along two days after 2^{nd} Test.......

... 3^{rd} TEST using MIRAFI P 40........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	4715	25	25	17.5
2	80.6	55.3	5052	25	25	18
3	80.6	55.3	5289	25	25	18
4	80.6	55.3	5847	25	25	18
Average	80.6	55.3	5226	25	25	17.9

$k_{ave} = 2.054 \times 10^{-6}$ cm/sec.

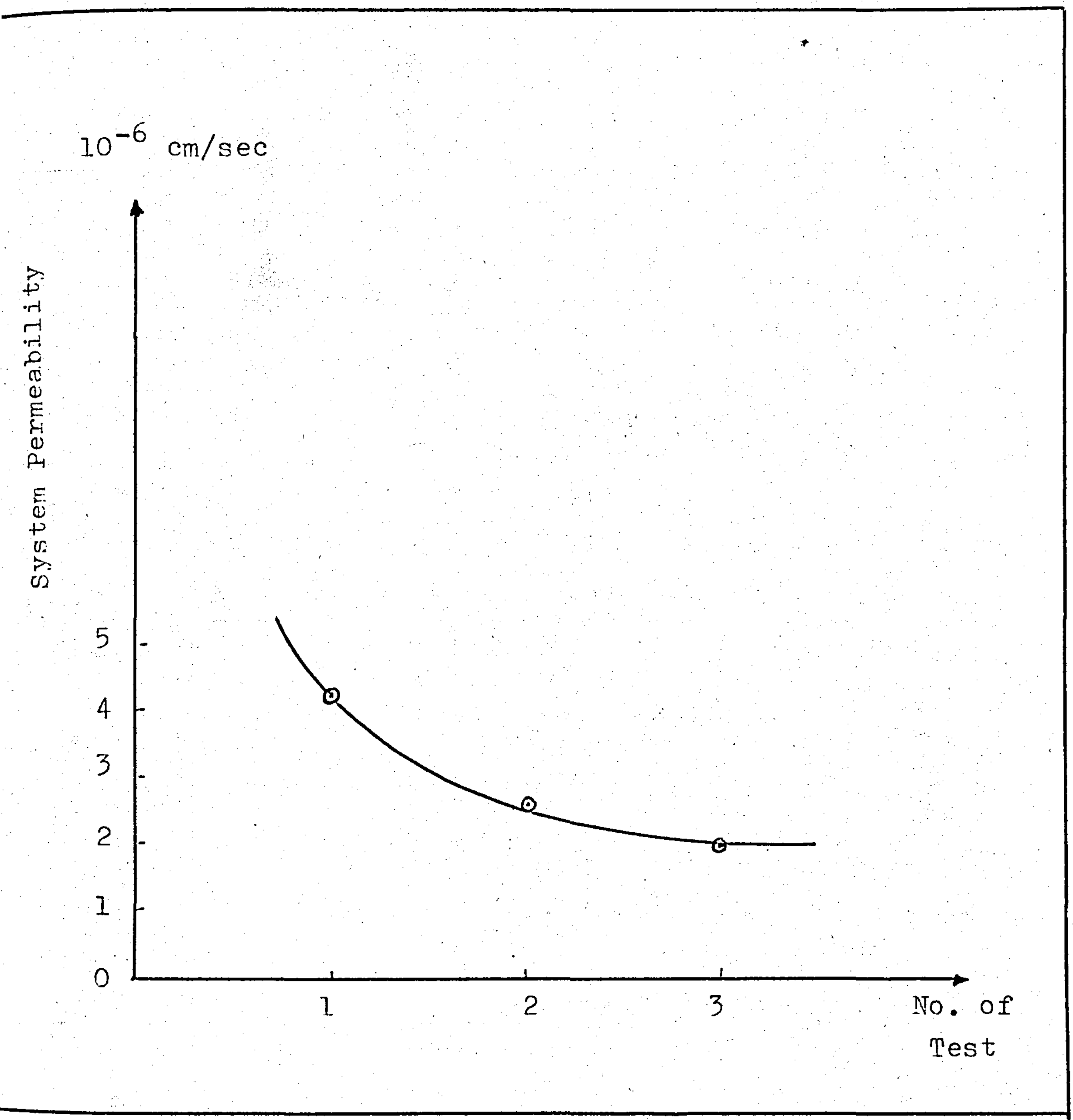


FIG. 6.9 NO OF TEST- SYSTEM PERMEABILITY CURVE for (Esan clay + MIRAFI P40 + Gravel Filter) system.

6.7.13. For (SATURATED BROWN CLAY + TYPAR 3207 + WET GRAVEL FILTER) SYSTEM.

Used standpipe = 50 ml. burette.

Cross-sectional area of the used burette, a = 0.988 cm^2.

Length of (Saturated Clay + Geotextile) layer. L = 5 cm.

...Using Virgin Geotextile.........

....1^{st} TEST using TYPAR 3207..........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	2682	25	25	14
2	80.6	55.3	2846	25	25	14
3	80.6	55.3	2991	25	25	13.8
4	80.6	55.3	3165	25	25	14.2
Average	80.6	55.3	2921	25	25	14

k_{ave} = 4.109 x 10^{-6} cm/sec.

6.7.14. For (SATURATED BROWN CLAY + TYPAR 3207 + WET GRAVEL FILTER) System.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a= o.988 cm^2.

Length of (Saturated clay + geotextile) layer, L= 5 cm.

...Using geotextile which is dried along two days after 1st Test.......

...2nd TEST using TYPAR 3207........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	4443	25	25	13.8
2	80.6	55.3	4817	25	25	14
3	80.6	55.3	5146	25	25	13.8
4	80.6	55.3	5251	25	25	13.6
Average	80.6	55.3	4914	25	25	13.8

k_{ave} = 2.429 x 10^{-6} cm/sec.

6.7.15 For (SATURATED BROWN CLAY + TYPAR 3207 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a = o.988 cm^2.

Length of (Saturated clay + geotextile) layer, L = 5 cm.

...Using geotextile which is dried along two days after 2nd Test........

...3rd TEST using TYPAR 3207........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	6543	25	25	14
2	80.6	55.3	7156	25	25	14.2
3	80.6	55.3	7441	25	25	14.2
4	80.6	55.3	7904	25	25	14
Average	80.6	55.3	7261	25	25	14.1

k_{ave} = 1.63 x 10^{-6} cm/sec.

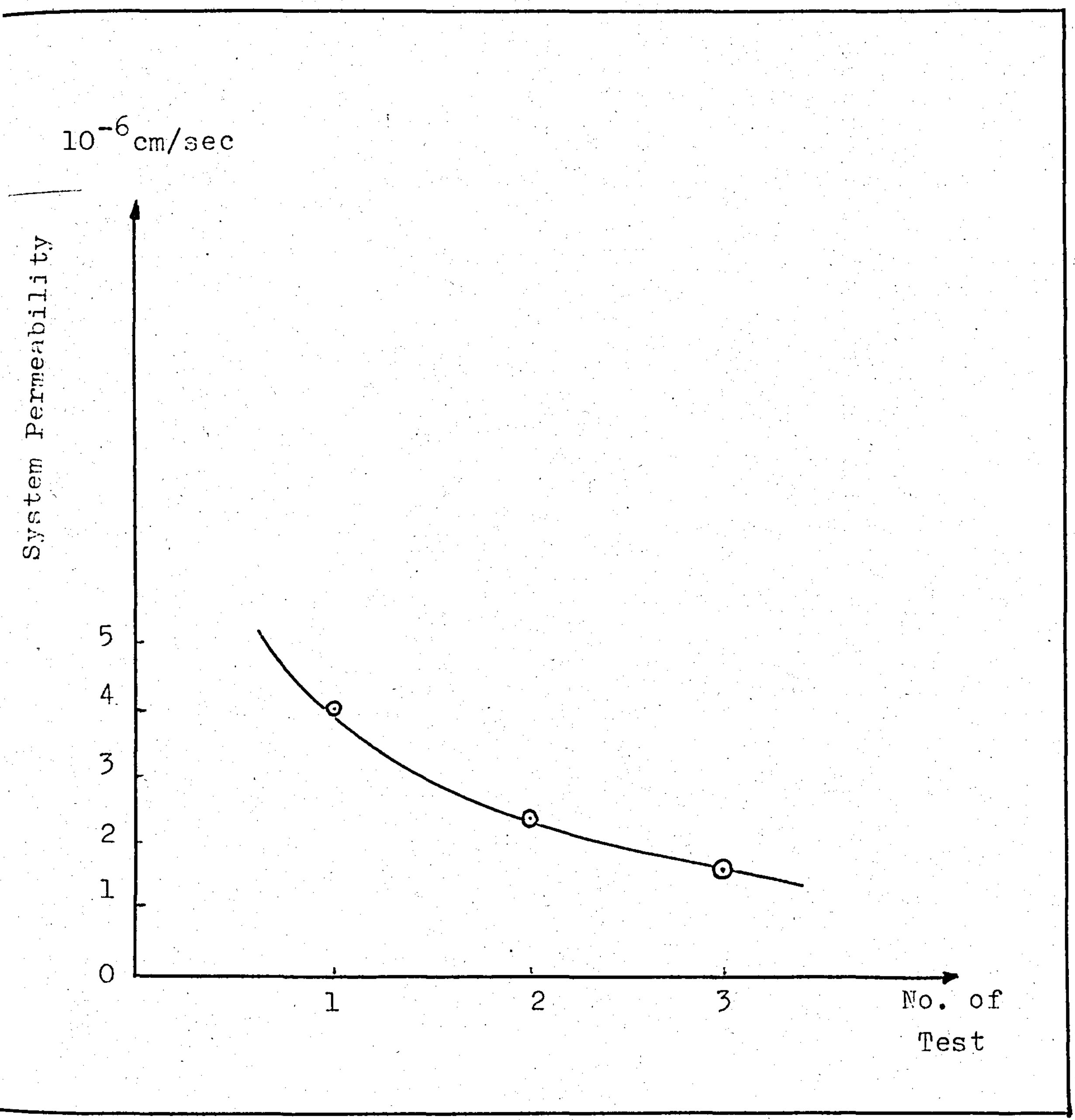


FIG. 6.10 NO OF TEST - SYSTEM PERMEABILITY CURVE
for (Brown clay + TYPAR 3207 + Wet gravel Filter)system.

6.7.16. For (SATURATED BROWN CLAY + TYPAR 3407-2 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette. a = o.988 cm^2.

Length of (Saturated clay + geotextile) layer, L = 5 cm.

...Using Virgin Geotextile.........

...1st TEST using TYPAR 3407-2........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	5780	25	25	13.5
2	80.6	55.3	6130	25	25	14
3	80.6	55.3	6526	25	25	14
4	80.6	55.3	7043	25	25	13.5
Average	80.6	55.3	6370	25	25	13.75

$k_{ave} = 1.877 \times 10^{-6}$ cm/sec.

6.7.17. For (SATURATED BROWN CLAY + TYPAR 3407-2 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a = o.988 cm^2.

Length of (Saturated clay + geotextile) layer, L = 5 cm.

...Using geotextile which is dried along two days after 1^{st} Test.........

...2^{nd} TEST using TYPAR 3407-2..........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	10422	25	25	14
2	80.6	55.3	11166	25	25	14
3	80.6	55.3	14170	25	25	14.5
4	80.6	55.3	16193	25	25	14
Average	80.6	55.3	12988	25	25	14.1

k_{ave} = 9.118 x 10^{-7}cm/sec.

6.7.18. For (SATURATED BROWN CLAY + TYPAR 3407-2 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a = 0.988 cm^2.

Length of (Saturated clay + geotextile) layer, L = 5 cm.

...Using geotextile which is dried along two days after 2nd Test.........

...3rd TEST using TYPAR 3407-2..........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	17043	25	25	14.5
2	80.6	55.3	18760	25	25	14.5
3	80.6	55.3	19188	25	25	14.5
4	80.6	55.3	21761	25	25	14.7
Average	80.6	55.3	19188	25	25	14.55

k_{ave} = 6.099 x 10^{-7} cm/sec.

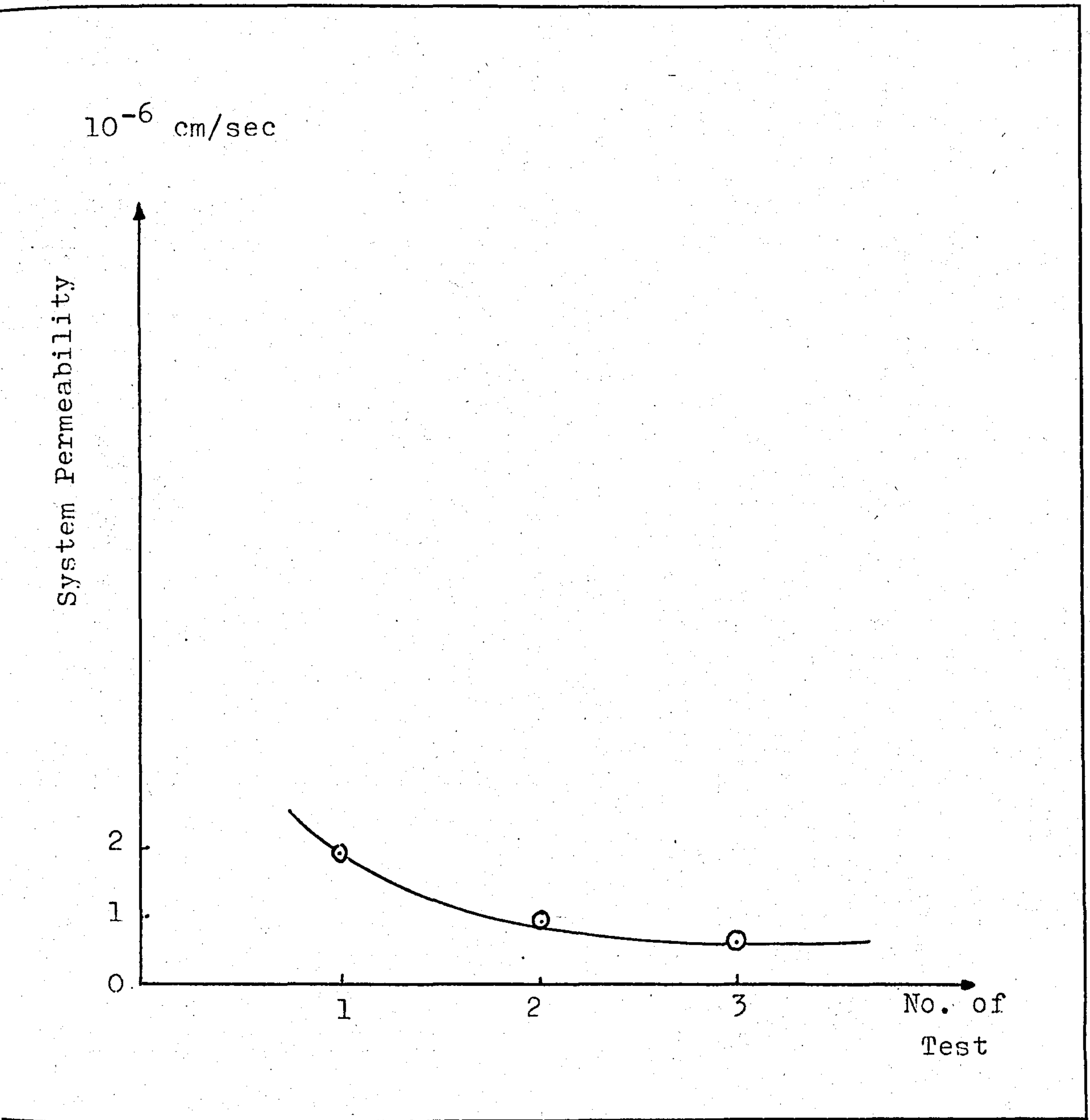


FIG. 6.11 NO OF TEST - SYSTEM PERMEABILITY CURVE for (Brown clay + TYPAR 3407-2 + Wet Gravel Filter)system.

6.7.19 For (SATURATED BROWN CLAY + TYPAR 3807-4 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a = o.988 cm^2.

Length of (Saturated clay + geotextile) layer, L = 5 cm.

...Using Virgin Geotextile........

...1st TEST using TYPAR 3807-4..........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	10134	25	25	12
2	80.6	55.3	11340	25	25	12.5
3	80.6	55.3	12334	25	25	12.5
4	80.6	55.3	13430	25	25	13
Average	80.6	55.3	11810	25	25	12.5

$k_{ave} = 1.047 \times 10^{-6}$ cm/sec.

6.7.20. For (SATURATED BROWN CLAY + TYPAR 3807-4 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a = o.988 cm^2.

Length of (Saturated clay + geotextile)layer, L = 5 cm.

...Using geotextile which is dried along two days after 1st Test..........

...2nd TEST using TYPAR 3807-4..........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	14022	25	25	14.5
2	80.6	55.3	14134	25	25	15
3	80.6	55.3	14847	25	25	14.5
4	80.6	55.3	15412	25	25	14.5
Average	80.6	55.3	14604	25	25	14.6

$k_{ave} = 8.003 \times 10^{-7}$ cm/sec.

6.7.21 For (SATURATED BROWN CLAY + TYPAR 3807-4 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a = 0.988 cm^2.

Length of (Saturated clay + geotextile)layer, L = 5 cm.

...Using geotextile which is dried along two days after 2nd Test.........

...3rd TEST using TYPAR 3807-4.........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	18106	25	25	14.8
2	80.6	55.3	19443	25	25	15
3	80.6	55.3	20417	25	25	14.9
4	80.6	55.3	22716	25	25	14.9
Average	80.6	55.3	20170	25	25	14.9

$k_{ave} = 5.748 \times 10^{-7}$ cm/sec.

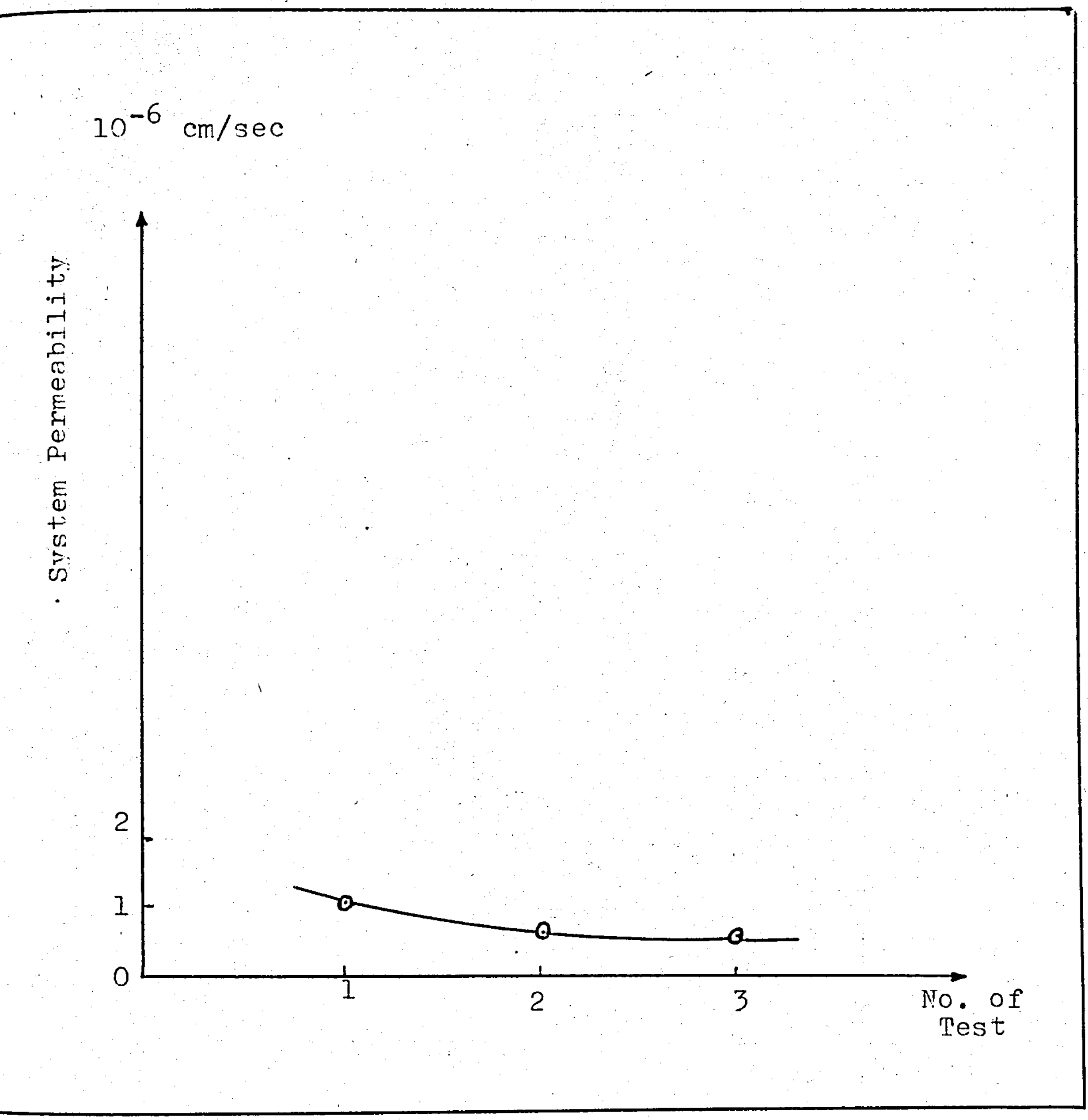


FIG. 6.12 NO. OF TEST - SYSTEM PERMEABILITY CURVE for (Brown clay + TYPAR 3807-4 + Wet Gravel Filter)system.

6.7.22. For(SATURATED BROWN CLAY + MIRAFI P40 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a = 0.988 cm^2.

Length of (Saturated clay + geotextile) layer, L = 5 cm.

...Using Virgin Geotextile.........

...1st TEST using MIRAFI P40............

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	3742	25	25	15
2	80.6	55.3	4183	25	25	15.5
3	80.6	55.3	5142	25	25	15
4	80.6	55.3	6681	25	25	16
Average	80.6	55.3	4937	25	25	15.4

$k_{ave} = 2.318 \times 10^{-6}$ cm/sec.

6.7.23. For (SATURATED BROWN CLAY + MIRAFI P40 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a = 0.988 cm^2.

Length of (Saturated clay + geotextile) layer, L = 5 cm.

...Using geotextile which is dried along two days after 1st Test...........

...2nd TEST using MIRAFI P40..........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	7151	25	25	16.5
2	80.6	55.3	7863	25	25	17
3	80.6	55.3	8176	25	25	17.2
4	80.6	55.3	9971	25	25	17
Average	80.6	55.3	8290	25	25	16.9

$k_{ave} = 1.328 \times 10^{-6}$ cm/sec.

6.7.24. For (SATURATED BROWN CLAY + MIRAFI P40 + WET GRAVEL FILTER) system.

Used standpipe = 50 ml. burette.

Cross-sectional area of the burette, a = o.988 cm^2.

Length of (Saturated clay + geotextile) layer, L = 5 cm.

,..Using geotextile which is dried along two days after 2^{nd} Test.........

...3^{rd} TEST Using MIRAFI P40..........

Test no	h_1	h_2	t	Q_{in}	Q_{out}	T
Unit	cm	cm	sec	cm^3	cm^3	^oC
1	80.6	55.3	13481	25	25	12.5
2	80.6	55.3	14383	25	25	13
3	80.6	55.3	16151	25	25	13.5
4	80.6	55.3	17240	25	25	13
Average	80.6	55.3	15316	25	25	13

k_{ave} = 7.963 x 10^{-7} cm/sec.

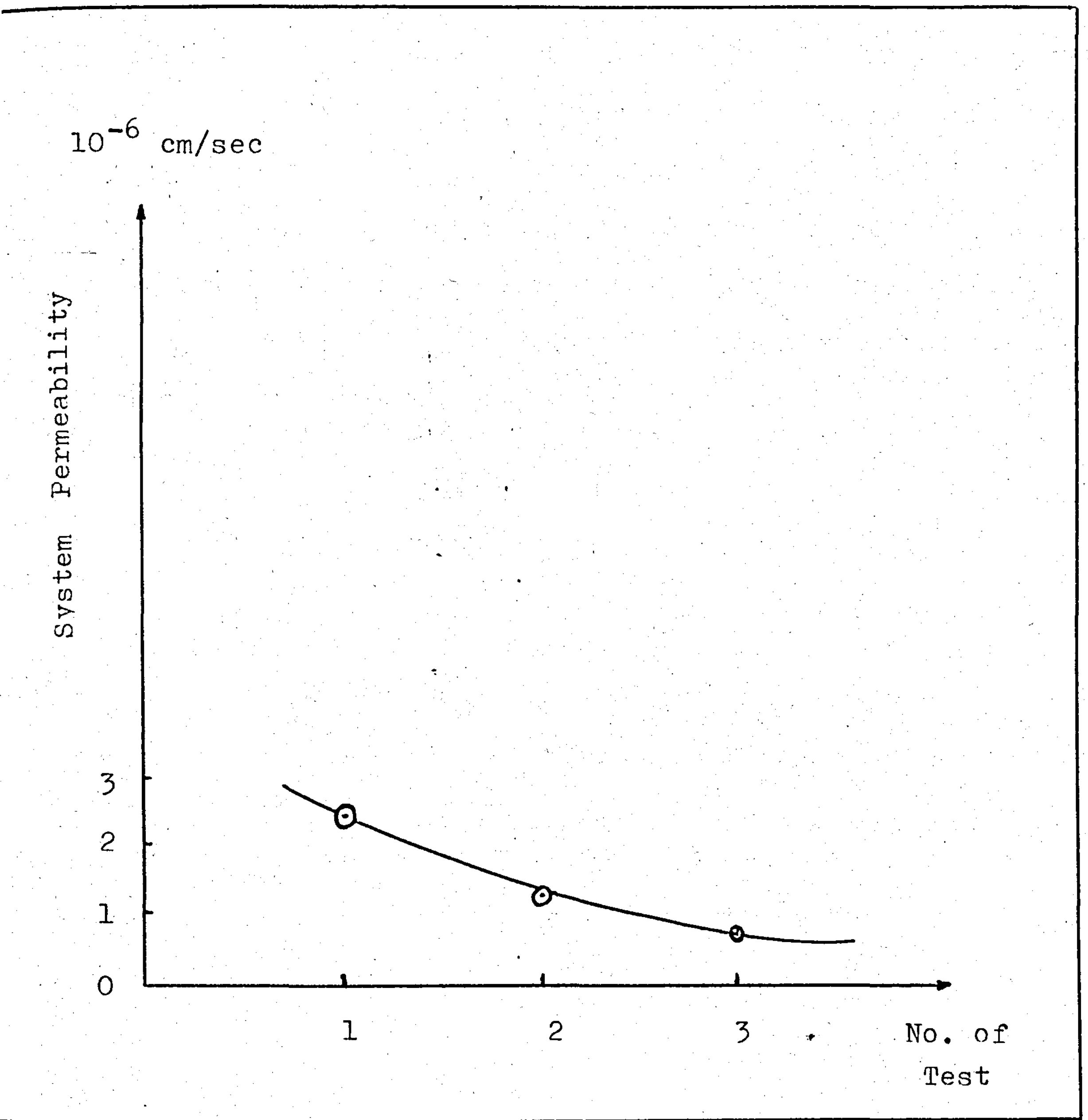


FIG. 6.13 NO. of TESTS - SYSTEM PERMEABILITY CURVE for (Brown clay + MIRAFI P40 + Wet gravel filter) system.

6.7.25. Tests for determining the clogging ratio.

The same permeameter has been used for this purpose. Firstly, the virgin geotextile has been placed between the two parts of the mold, and then test is performed. Later, the clogged geotextiles has been subjected to same tests. 50 ml. water has been allowed to flow through the geotextile and elapsed time has been recorded. The cross-sectional area of geotextile is known. Using the following Equation, the flow through the geotextile may be determined:

$$k_f = \frac{Q}{A\,t}$$

Where,

k_f = flow through the geotextile, in $1/m^2$sec.

Q = amount of water flowing through the geotextile, in 1.

A = cross-sectional area of the geotextile, in m^2.

t = elapsed time, in sec.

6.7.25.1. Clogging Ratio test on TYPAR 3207 which has been tested with Esan Yellow Clay.

	Virgin Geotextile			Clogged Geotextile		
Test no	Q	A	t	Q	A	t
Unit	10^{-3} l.	10^{-4} m^2	sec	10^{-3} l.	10^{-4} m^2	sec
1	50	182.415	18	50	182.415	29
2	50	182.415	18.5	50	182.415	31
3	50	182.415	18	50	182.415	30
Ave.	50	182.415	18.2	50	182.415	30

$$k_{f1} = \frac{50 \times 10^{-3}\ l}{(182.415 \times 10^{-4}\ m^2)(18.2\ sec)}$$

$$k_{f1} = 0.151\ \ l/m^2sec.$$

$$k_{f2} = \frac{(50 \times 10^{-3}\ l)}{(182.415 \times 10^{-4} m^2)(30\ sec)}$$

$$k_{f2} = 0.091\ \ l/m^2sec.$$

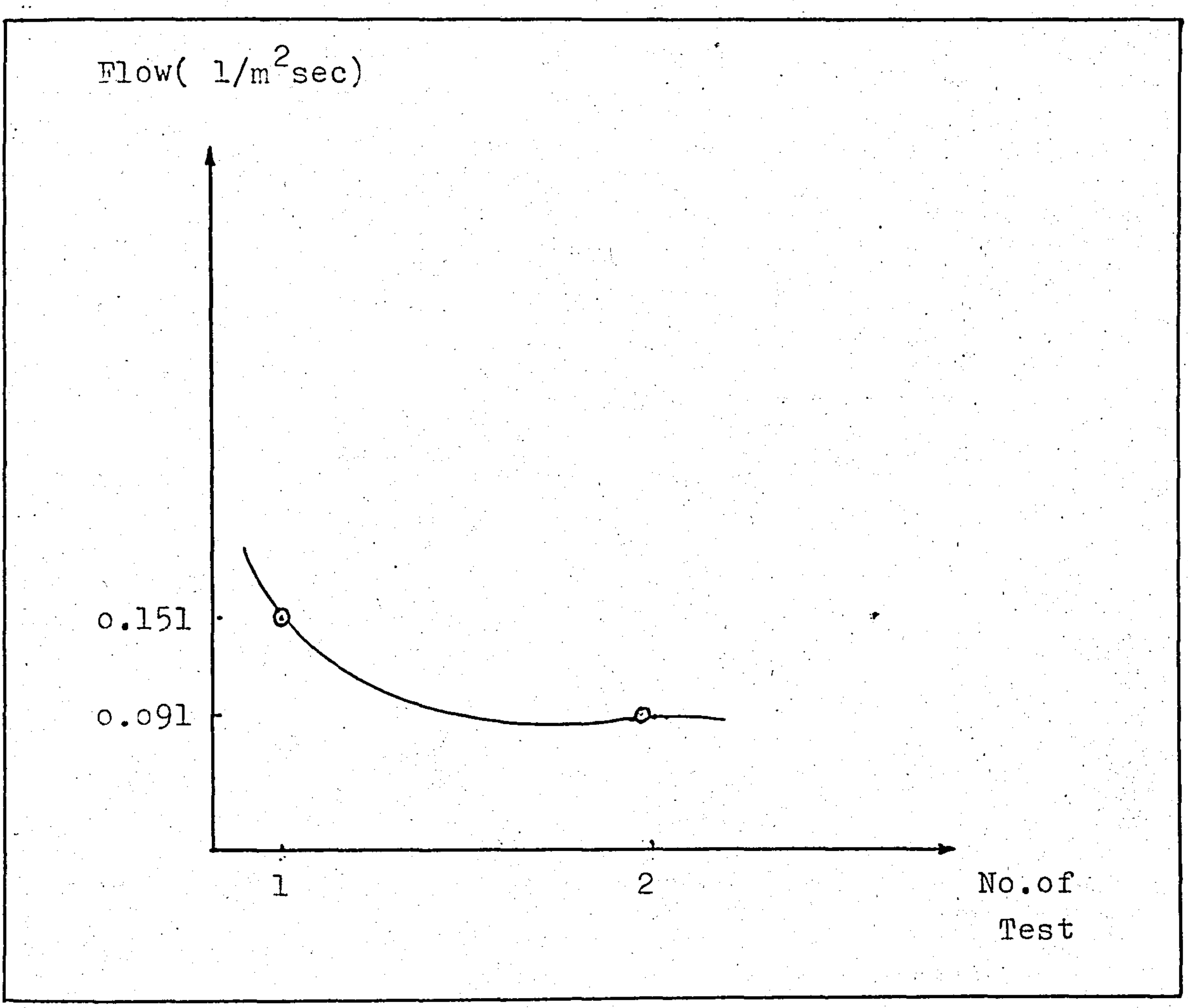


FIG.6.14. VARIATION OF FLOW THROUGH GEOTEXTILE WITH NUMBER OF TEST. (For TYPAR 3207)

1 indicates the flow of the virgin TYPAR 3207

2 indicates the flow of the clogged TYPAR 3207 which has been tested with Esan Clay

6.7.25.2. Clogging Ratio Test on TYPAR 3207 which has been tested with Esan Yellow Clay.

	Virgin Geotextile			Clogged Geotextile		
Test no	Q	A	t	Q	A	t
Unit	10^{-3} l.	$10^{-4}m^2$.	sec	10^{-3}l.	$10^{-4}m^2$.	sec
1	50	182.415	18	50	182.415	39
2	50	182.415	18.5	50	182.415	41
3	50	182.415	18	50	182.415	40
Ave.	50	182.415	18.2	50	182.415	40

k_{f1} = 0.151 $1/m^2$ sec.

k_{f2} = 0.069 $1/m^2$sec.

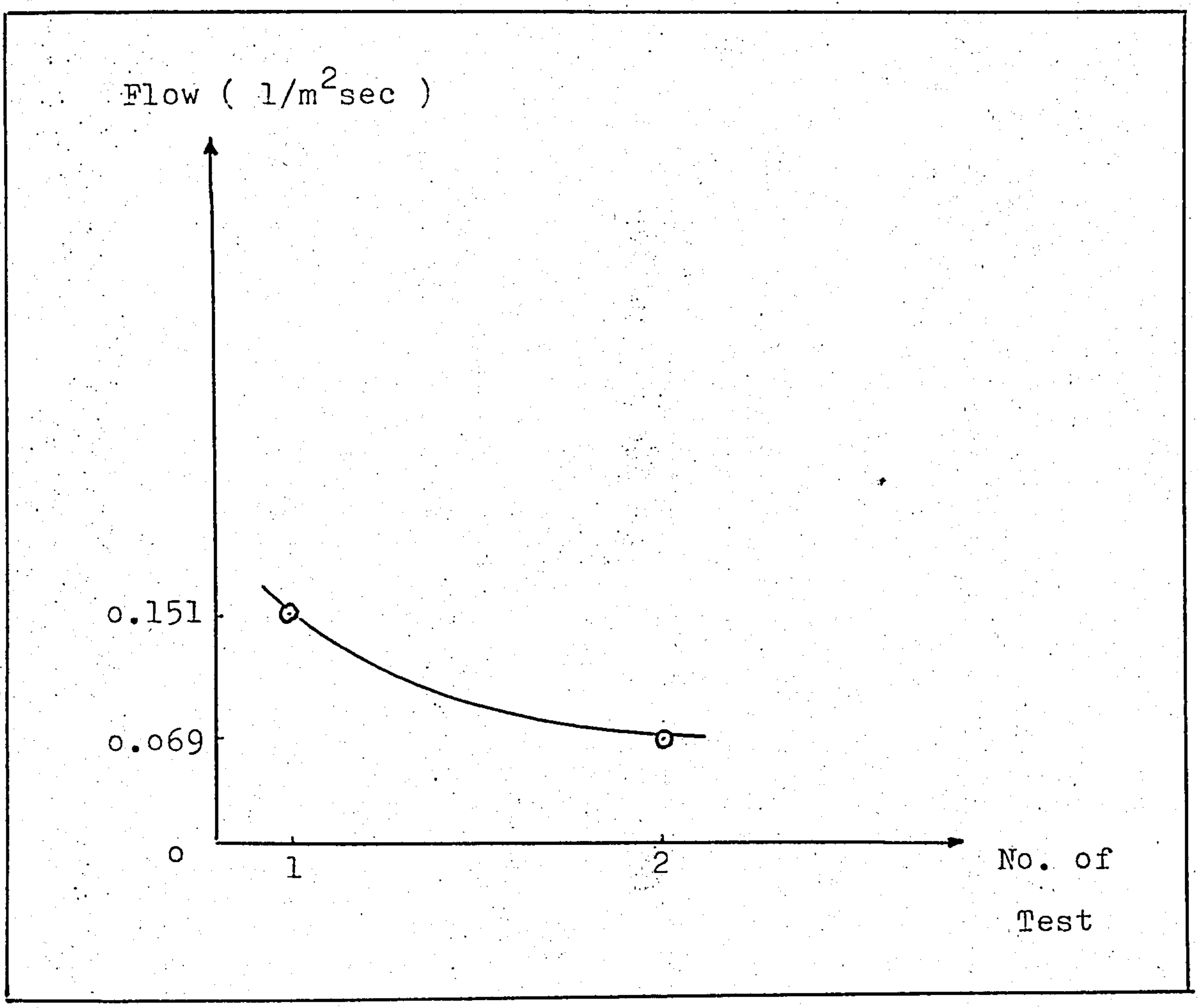


FIG.6.15. VARIATION OF FLOW THROUGH GEOTEXTILE WITH NUMBER OF TEST. (For TYPAR 3207).

1 indicates the flow of the virgin TYPAR 3207

2 indicates the flow of the clogged TYPAR 3207 which has been tested with Brown Clay.

6.7.25.3. Clogging Ratio Test on TYPAR 3407-2 which has been tested with Esan Yellow Clay.

	Virgin Geotextile			Clogged Geotextile		
Test no	Q	A	t	Q	A	t
Unit	10^{-3} l.	$10^{-4}m^2$	sec	10^{-3} l.	$10^{-4}m^2$.	sec
1	50	182.415	49	50	182.415	122
2	50	182.415	51	50	182.415	125
3	50	182.415	50	50	182.415	125
Ave.	50	182.415	50	50	182.415	124

$$k_{f1} = \frac{50 \times 10^{-3} \text{ l.}}{(182.415 \times 10^{-4} m^2)(50 \text{ sec})}$$

$$k_{f1} = 0.055 \quad 1/m^2\text{sec.}$$

$$k_{f2} = \frac{50 \times 10^{-3} \text{ l.}}{(182.415 \times 10^{-4} m^2)(124 \text{ sec})}$$

$$k_{f2} = 0.022 \quad 1/m^2\text{sec.}$$

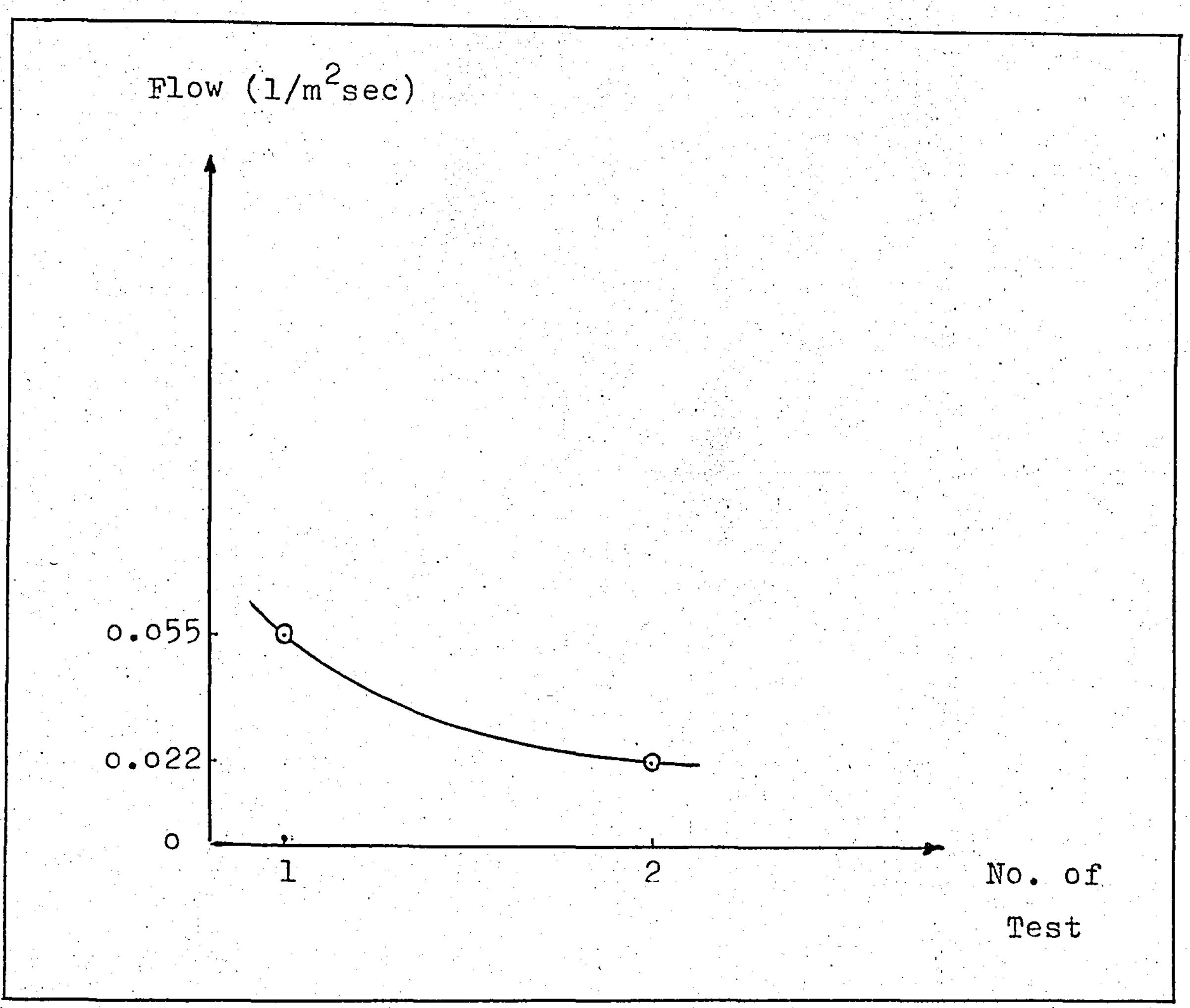


FIG.6.16. VARIATION OF FLOW THROUGH GEOTEXTILE WITH NUMBER OF TEST. (For TYPAR 3407-2)

1 indicates the flow of the virgin TYPAR 3407-2

2 indicates the flow of the clogged TYPAR 3407-2 which has been tested with Esan Clay.

6.7.254 Clogging Ratio Test on TYPAR 3407-2 which has been tested with Brown Clay.

	Virgin Geotextile			Clogged Geotextile		
Test no	Q	A	t	Q	A	t
Unit	10^{-3} l.	10^{-4} m^2.	sec.	10^{-3} l.	10^{-4} m^2.	sec.
1	50	182.415	49	50	182.415	146
2	50	182.415	51	50	182.415	151
3	50	182.415	50	50	182.415	157
Ave.	50	182.415	50	50	182.415	151

$k_{f1} = 0.055$ $1/m^2$sec.

$k_{f2} = 0.018$ $1/m^2$sec.

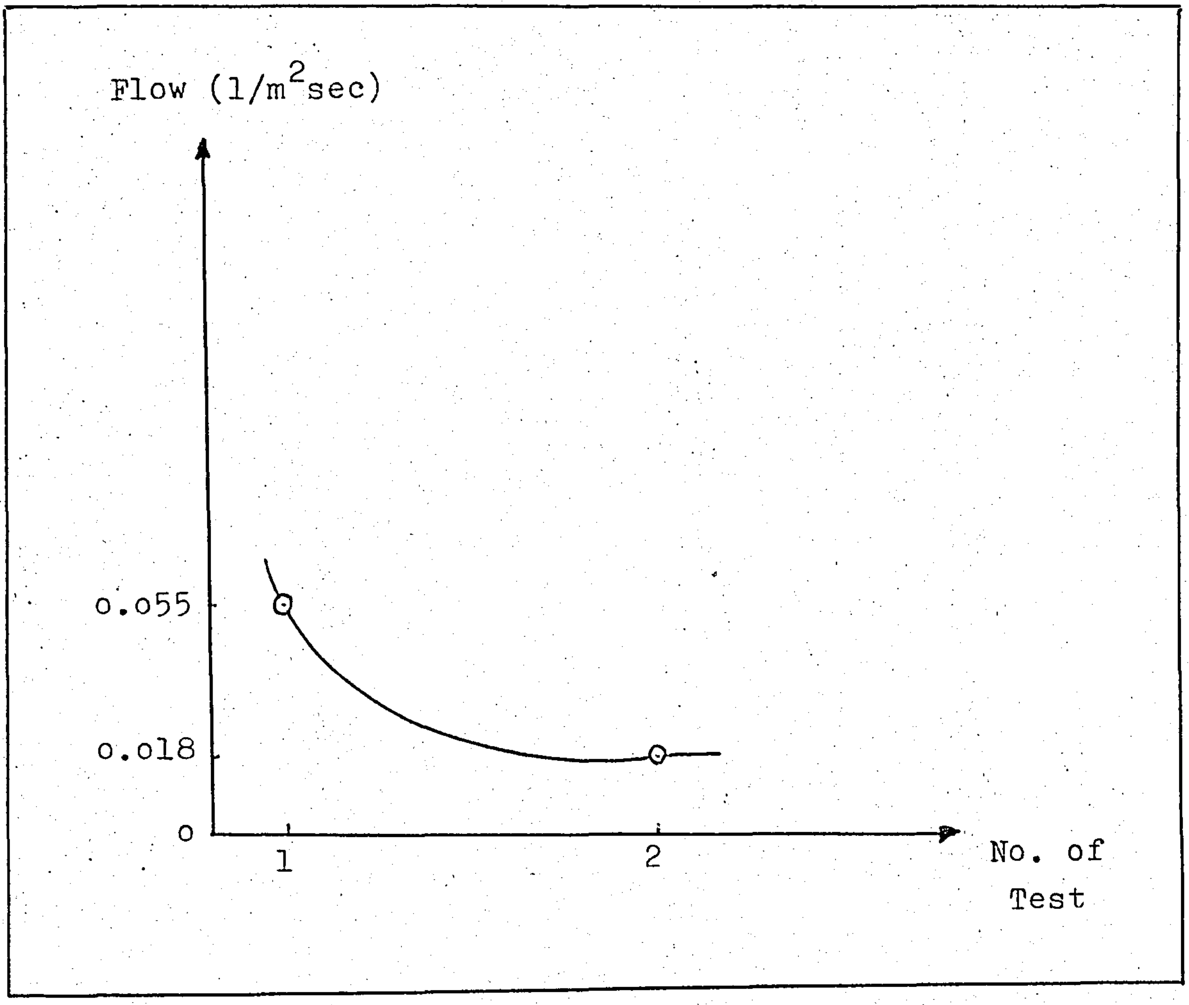


FIG.6.17. VARIATION OF FLOW THROUGH GEOTEXTILE WITH NUMBER OF TEST. (For TYPAR 3407-2)

1 indicates the flow of the virgin TYPAR 3407-2

2 indicates the flow of the clogged TYPAR 3407-2 which has been tested with Brown Clay.

6.7.25.5. Clogging Ratio Test on TYPAR 3807-4 which has been tested with Esan Yellow Clay.

	Virgin Geotextile			Clogged Geotextile		
Test no	Q	A	t	Q	A	t
Unit	10^{-3} l.	10^{-4} m^2	sec	10^{-3} l.	$10^{-4}m^2$	sec
1	5o	182.415	67	50	182.415	166
2	5o	182.415	71	5o	182.415	185
3	5o	182.415	8o	5o	182.415	193
Ave.	5o	182.415	73	5o	182.415	181

$$k_{f1} = \frac{(5o \times lo^{-3}\ l.)}{(182.415 \times lo^{-4}\ m^2)(73)}$$

$$k_{f1} = o.o38\ \ 1/m^2sec.$$

$$k_{f2} = \frac{(5o \times lo^{-3}\ l.)}{(182.415 \times lo^{-4}\ m^2)(181)}$$

$$k_{f2} = o.o15\ \ 1/m^2sec.$$

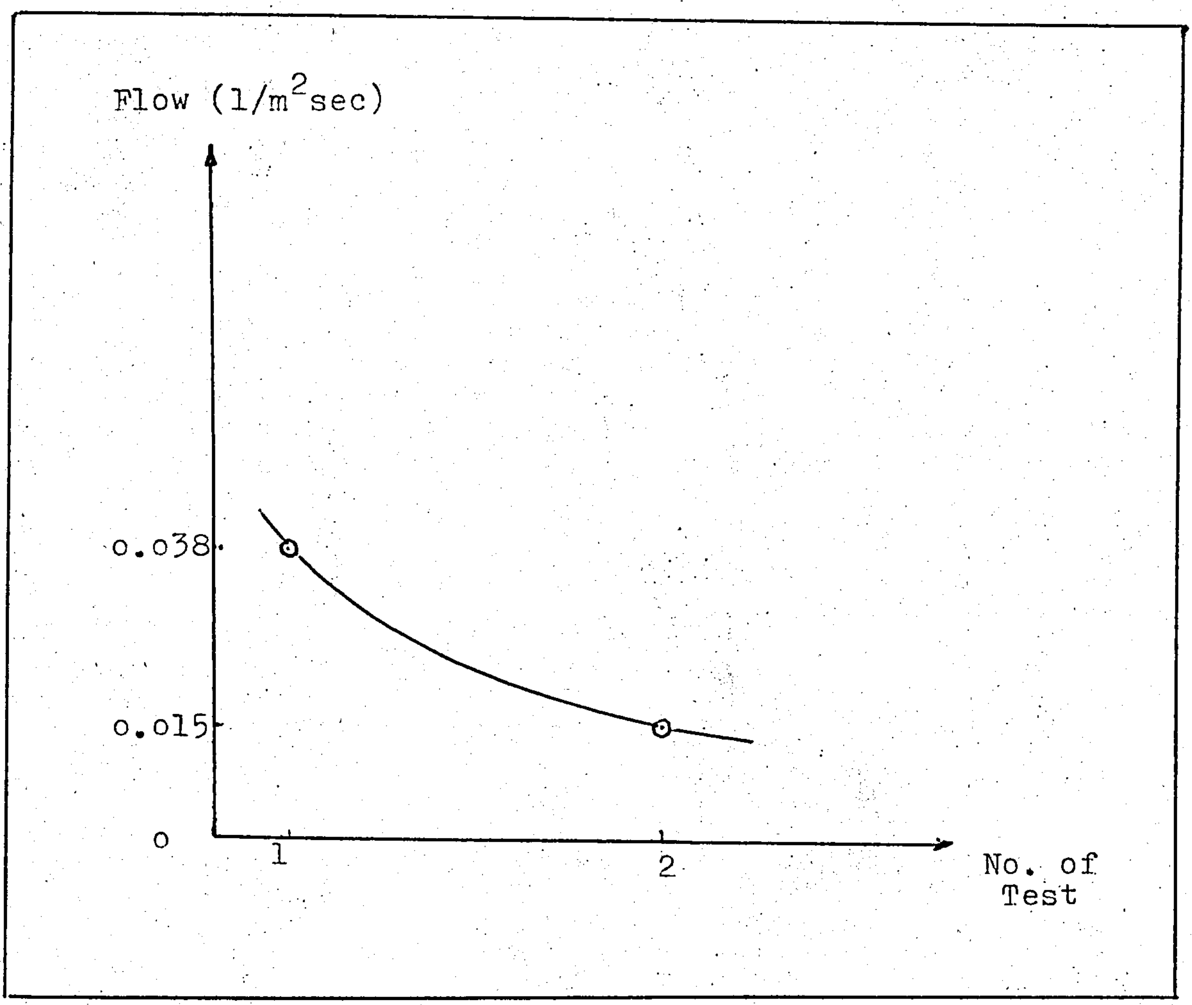


FIG.6.18. VARIATION OF FLOW THROUGH GEOTEXTILE WITH NUMBER OF TEST. (For TYPAR 3807-4)

1 indicates the flow of the virgin TYPAR 3807-4

2 indicates the flow of the clogged TYPAR 3807-4 which has been tested with Esan Yellow Clay.

6.7.25.6. Clogging Ratio Test on TYPAR 3807-4 which has been tested with Brown Clay.

	Virgin Geotextile			Clogged Geotextile		
Test no	Q	A	t	Q	A	t
Unit	10^{-3} l	$10^{-4}m^2$	sec	10^{-3} l	$10^{-4}m^2$	sec
1	50	182.415	67	50	182.415	218
2	50	182.415	71	50	182.415	223
3	50	182.415	80	50	182.415	226
Ave.	50	182.415	73	50	182.415	222

$k_{f1} = 0.038 \quad 1/m^2sec.$

$k_{f2} = 0.012 \quad 1/m^2sec.$

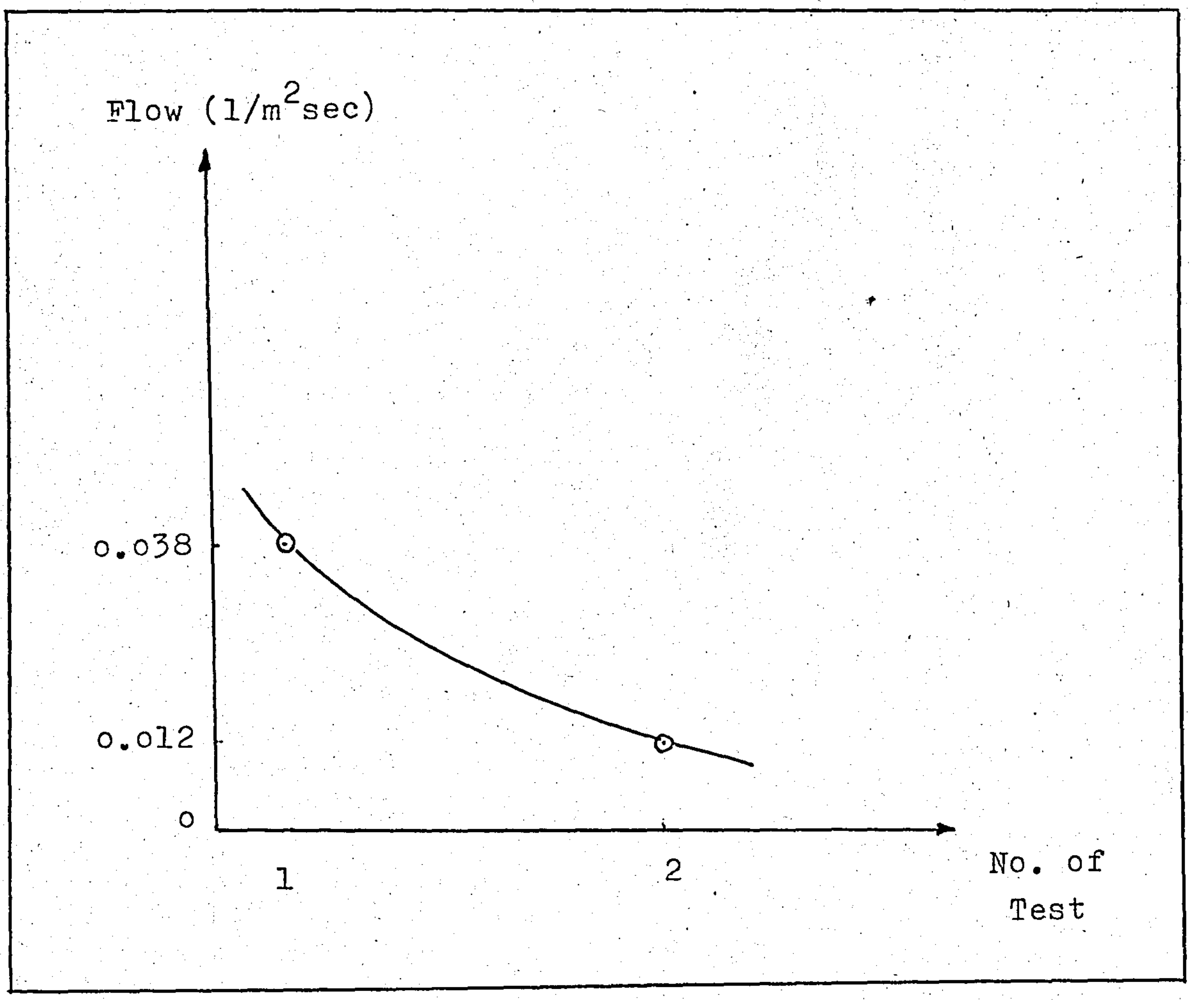


FIG.6.19. VARIATION OF FLOW THROUGH GEOTEXTILE WITH NUMBER OF TEST (For TYPAR 3807-4).

1 indicates the flow of the virgin TYPAR 3807-4

2 indicates the flow of the clogged TYPAR 3807-4 which has been tested with Brown Clay

6.7.25.7. Clogging Ratio Test on MIRAFI P40 which has been tested with Esan Yellow Clay.

	Virgin Geotextile			Clogged Geotextile		
Test no	Q	A	t	Q	A	t
Unit	10^{-3} l	$10^{-4}m^2$	sec	10^{-3} l	$10^{-4}m^2$	sec
1	50	182.415	27	50	182.415	63
2	50	182.415	29	50	182.415	67
3	50	182.415	35	50	182.415	71
Ave.	50	182.415	30	50	182.415	67

$$k_{f1} = \frac{(50 \times 10^{-3}\ \text{l.})}{(182.415 \times 10^{-4} m^2)(\ 30\ \text{sec})}$$

$$k_{f1} = 0.091\ \ \text{l}/m^2\text{sec.}$$

$$k_{f2} = \frac{(50 \times 10^{-3}\ \text{l.})}{(182.415 \times 10^{-4}\ m^2)(\ 67\ \text{sec.})}$$

$$k_{f2} = 0.041\ \ \text{l}/m^2\text{sec.}$$

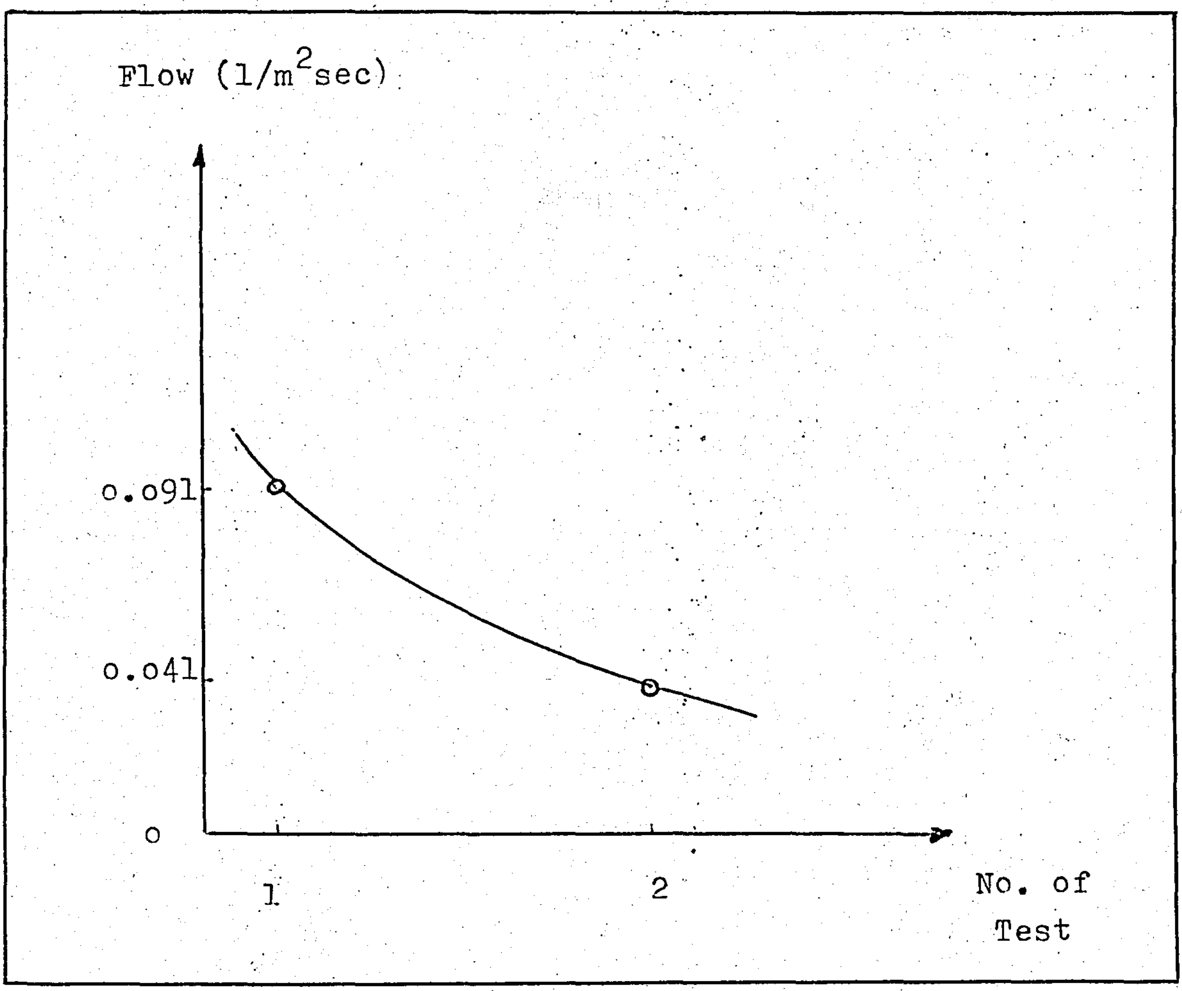


FIG.6.20. VARIATION OF FLOW THROUGH GEOTEXTILE WITH NUMBER OF TEST (For MIRAFI P40).

1 indicates the flow of the virgin MIRAFI P40

2 indicates the flow of the clogged MIRAFI P40 which has been tested with Esan Yellow Clay

6.7.25.8. Clogging Ratio Test on MIRAFI P40 which has been tested with Brown Clay.

	Virgin Geotextile			Clogged Geotextile		
Test no	Q	A	t	Q	A	t
Unit	10^{-3} l.	10^{-4} m^2	sec	10^{-3} l	$10^{-4}m^2$	sec
1	50	182.415	27	50	182.415	81
2	50	182.415	29	50	182.415	86
3	50	182.415	35	50	182.415	82
Ave.	50	182.415	30	50	182.415	83

k_{f1} = 0.091 l/m^2sec.

k_{f2} = 0.033 l/m^2 sec.

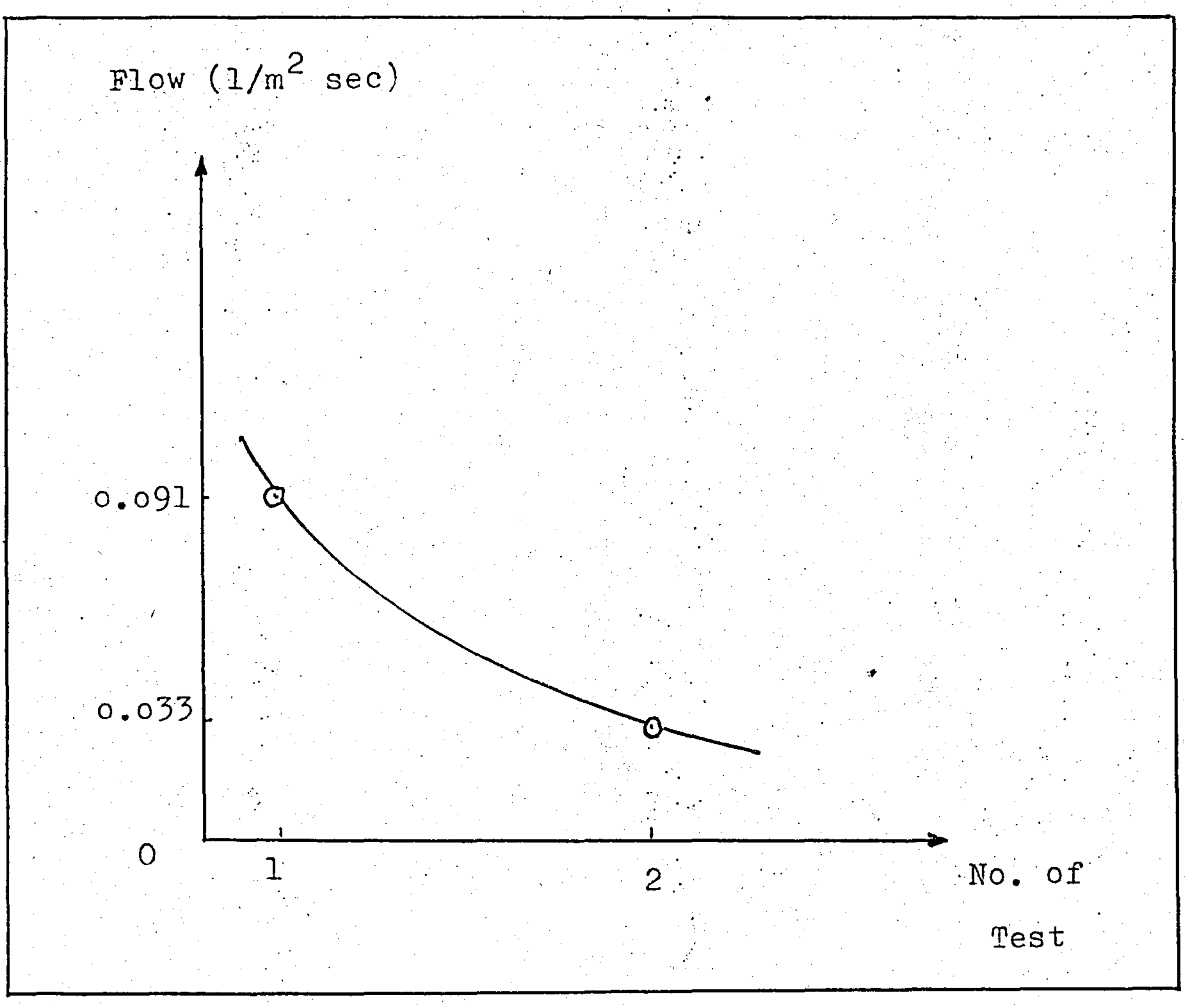


FIG.6.21. VARIATION OF FLOW THROUGH GEOTEXTILE WITH NUMBER OF TEST (For MIRAFI P40).

1 indicates the flow of the virgin MIRAFI P40

2 indicates the flow of the clogged MIRAFI P40 which has been tested with Brown Clay

CHAPTER 7

CONCLUSIONS

In order to better understand the clogging phenomena in the soil filter systems, several water permeability tests were performed for (Saturated Clay + Geotextile + Wet Gravel Filter) systems.

The following conclusions can be derived from this study:

1. At the begining, the permeability of(saturated clay + geotextile + wet gravel filter) system is closely near to the permeability of the saturated clay. After the geotextile is dried and re-tested, it may be seen that there is a decrease in the permeability of the system due to clogging. If the geotextile is re-dried and tested again, the permeability of the system is remained approximately constant. This shows that the permeability of the system reaches an equilibrium point.

2. The rate of clogging phenomena for geotextile which has greater percent open area is smaller than the rate of clogging for geotextile which has smaller percent open area. It may be concluded that the greater a geotextile's percent open area, the greater its resistance to clogging.

3.The clogging phenomena is dependent on the particle size of the silty soil.For example,for (Saturated Esan Yellow Clay + TYPAR 3207 + Wet Gravel Filter) system,both permeability and piping requirements are satisfied,due to these reasons, there is no cldgging.For (Saturated Brown Clay + TYPAR 3207 + Wet Gravel Filter) system,permeability requirement is satisfied , but piping requirement is not satisfied,thus there is clogging.

4.The clogging phenomena is also dependent on the plasticity of the soil.If the soil has high plasticity,the clogging will be large.

R E F E R E N C E S

1.ANDRAWES,K.Z.,McGOWN,A.,and WILSON-FAHMY,D.F., "The Behaviour of a Geotextile Reinforced Sand Loaded by a Strip Footing",Proceedings of the 8th European Conference on Soil Mechanics and Foundation Engineering, Vol.2,1983,pp.329-334.

2.BATERAU,C.,and PAUL.O.,"Improvement of Ground Through The Application of Geotextiles",Proceedings of the 8th European Conference on Soil Mechanics and Foundation Engineering,Vol.2,1983,pp.459-462.

3.BOUTRUP,E.,HOLTZ,R.D.,"Analysis of Embankments on Soft Ground Reinforced with Geotextiles",Proceedings of the 8th European Conference on Soil Mechanics and Foundation Engineering,Vol.2,1983,pp,469-472.

4.CARROLL,R.G.,"Haliburton Gradient Ratio Testing",Mirafi Technical Note,1983.

5.CEDERGREN,H.R.,"Seepage,Drainage and Flow Nets", John Wiley and Sons,1977.

6.CROWFORT,P.,INGOLD,T.S.,"The Notion of Geotextiles as Separators in Roads",Journal of the Ground Engineering, No.1,1984,pp.27-34.

7.GOURC,J.P.,PERRIER,H.,RIONDY,G.,"Unsurfaced Roads on Soft Subgrade:Mechanism of Geotextile Reinforcement",Proceedings of the 8th European Conference on Soil Mechanics and Foundation Engineering,Vol.2,1983,pp.495-498.

8.HEERTEN,G.,"Geotextiles in Coastal Engineering",Journal of Geotextiles and Geomembranes,No.1,1984,pp.119-141.

9.HOARE,D.J.,"Geotextiles As Filters",Journal of the Ground Engineering,No.2,1984,pp.29-44.

10.HOARE,D.J.,"Synthetic Fabrics as Soil Filters",Journal of the Geotechnical Engineering Division,Proceedings of the American Society of Civil Engineers,Vol.108,No.GT10, October 1982,pp.1230-1245.

11.HOARE,D.J.,"Geotextiles in the UK",Journal of the Ground Engineering,No.5,1983,pp. 30-38.

12.INGOLD,T.S.,"A Laboratory Investigation of Soil-Geotextile Friction",Journal of the Ground Engineering,No.8, 1984, pp.647-653.

13.INGOLD,T.S.,"Some Factors in the Design of Geotextile Reinforced Embankments",Proceedings of the 8th European Conference on Soil Mechanics and Foundation Engineering Vol.2,1983,pp.503-508.

14. KRYNINE,D.P.,and JUDD,W.R.,"Principles of Engineering Geology and Geotechnics",1957,McGraw-Hill Book Company,Inc.

15. LAMBE,T.W.," The Permeability of Fine-Grained Soils", ASTM Technical Publication,No 163,1954,pp. 56-67.

16. LAWSON,C.R.," Filter Criteria for Geotextiles:Relevance and Use",Journal of the Ground Engineering Division, Proceedings of the American Society of Civil Engineers, Vol.108,No GT10,October 1982,pp. 1300-1317,

17. QUAST,P.," Polyester Reinforçing Fabric Mats for the Improvement of Embankment Stability",Proceedings of the 8th European Conference on Soil Mechanics and Foundation Engineering,Vol.2,1983,pp. 531-534.

18. SCHOBER,W.,and TEINDL,H.," Filter Criteria for Geotextiles", Proceedings of the 7th European Conference on Soil Mechanics and Foundation Engineering,Vol.2,1979,pp.121-129.

19. SCOTT,R.F.,"Principles of Soil Mechanics",Addison-Wesley Publishing Company,Inc.,1963.

20. SHERARD,J.L.,DUNNIGAN,L.P.,and TALBOT,J.R.,"Basic Properties of Sand and Gravel Filters",Journal of the Ground Engineering Division,Proceedings of the American Society of Civil Engineers,Vol.110,No 6,June 1984,pp.684-700

21. SPANGLER,M.G.,"Soil Engineering",International Textbook Company,1966.