4.0 LABORATORY & FIELD INVESTIGATION

4.1 Survey and Sequences

A field reconnaissance of the drainage problem is one of the most important step in design consideration. The reconnaissance is usually an inspection of the area to get fully acquainted with the problem, topographic conditions and physical features. Parts of the watershed outside the drainage .problem area must be examined to determine runoff characteristics and land treatment required. The following points must be considered for field reconnaissance.

1. Determine the extent of the area needing improvement. Type of improvements required should be determined *i.e. surface drainage*, subsurface drainage, flood prevention.

2. Review of existing drainage system-location, spacing, type of drainage and depth.

3. Determine the adequacy of drainage outlets and its condition.

4. Topography and size of the drainage area.

5. Location and characteristics of existing canals, laterals, wells, ponds etc.

6. Local irrigation practices such as method of irrigation and efficiency of irrigation.

7. Type of crops currently grown, cropping practices, crop conditions and identification of the areas on which crop damage is suffered.

8. Water table fluctuation and direction of flow.

9. General characteristics of soil throughout the area needing drainage, identification of salinity and alkalinity spots.

10. Type of construction equipment available or availability of the labour and the prevailing rates of works.

4.1.1. Investigation for Surface Drainage

The various investigation usually required for planning and design of surface drainage system are

i) Topographic surveys

ii) Soil surveys
iii) Land use and cropping pattern
iv) Water sources survey
v) *Profile and cross sections of existing streams, and ditches*
vi) Stage-frequency investigation of high water in outlets. Geologic investigations.

i) Topographic Surveys

Topographic survey is essential for design of drainage system. It is the framework upon which soil survey, waterlevel survey, drain location and depth depends considerably. A topographic survey gives a due to the type of drainage needed and provide informations for specific drainage planning. The topographic map must show all physical features of watershed, both natural and manmade which affects design of drainage system. Physical features like location of ditches, roads, railroads, water courses, farm boundaries, land use, drainage boundaries, surface contours should be shown on the map.

Field survey are done in various ways depending upon the equipment available and detail required. The plaintable with telescopic alidade is excellent for obtaining detailed topography.

ii) Soil Investigation

For good drainage design, adequate information must be obtained on the characteristics of the soils to be drained. This information is obtained by examining the physical characteristics of the soil that relate to drainage design. The soil can best be examined by excavation or boring. If soil survey report for the project area is not available, detail soil investigation must be done and information on hydraulic conductivity, water table, salinity and productivity must be obtained.

Soil quality for drainage purposes can be classified into its physical and chemical properties. Physical properties include texture, density and structure. The texture class of a soil is determined by the size of soil particles and classes are identified by proportions of sand silt and clay. Soil structure indicates the arrangement of the soil particles. Generally the coarse textured soils drain better than the fine textured soils, although texture is not related necessarily to permeability. In most irrigated areas the soils are formed into complex profile pattern, stratified sands, silts and clays are commonly found. Sometime fine textured clay layers are underlain or overlain by coarse textured sands. This can be investigated by boring holes to find out where the coarse drainable layers are located. As the hole is bored, the various soil layers are recorded and their drainability is estimated. The sequence of permeable and impermeable soils and their ability to transmit water determines both the type of system-surface and subsurface, that should be installed and design. Soil chemical properties as well as physical properties must be considered in evaluating the soil quality for drainage purposes. If a soil has a high sodium content, application of irrigation water may cause deterioration of the soil structure and decrease the permeability to the point that both leaching and drainage will be affected. If evidence of salinity. Such as poor growth of plant, accumulation of salt on the surface or a very soft and dry condition on the surface is observed, soil sample should be taken to determine the chemical properties of the soil. Factors to be considered are electrical conductivity, sodium content, calcium content and pH of the soil.

iii) Land use and Cropping pattern

Land use data can be obtained from aerial photographs. The cropping pattern data on cultivated land is needed to select proper rates of drainage and to make economic evaluation. Although some crops are more adopted to shallow water tables than others, the idle drainage for most economic crop plants is unrestricted downward movement of excess water. If the water table is being controlled at a specific depth for the purpose of providing water to the plants, then variable such as rainfall, root depth, plant variety and soil texture must be considered. In high rainfall areas, the water table must be low enough to maintain soil aeration. Since crop rotations are practiced on many drained soils, the depth of water table in an artificially drained field should usually be designed to provide a water table depth satisfactory for the deepest rooted plants.

iv) Water sources study

The common sources of water in drainage problem areas are (1) precipitation (2) irrigation (3) seepage (4) hydrostatic pressure and (5) combination of any of these sources. The source of excess water which creates drainage problem must be determined so that proper measures can be taken. Water sources survey will also determine the type of drainage to be installed. Thus if excess water is due to precipitation, the solution may involve provision of surface drainage; if overirrigation, the solution may involve use of proper irrigation and water management practices; if due to canal seepage, the solution may involve canal leving and provision of interception drain may be indicated; and if due to artesian or hydrostatic pressure; pumped wells provide the most practicable remedy. The water sources which creates drainage problem are briefly discussed below.

v) Precipitation

Precipitation records must be collected and analysed in order to determine the distribution of rainfall, its frequency and cyclic trend. The distribution of precipitation must be related to water table fluctuation. A positive correlation between two may give the seasonal precipitation as Chief source of water causing drainage problem. A poor correlation indicates that precipitation probably has little effect on the water table. In humid area, precipitation is the main cause of drainage problems. Also long term records of precipitation should be related to long term hydrographs of water levels in order to determine whether the wet cycles are followed by rising water tables and vice versa.

vi) **Outfall-gauge records on long term basis are also needed for proper drainage design.**

4.1.2 Investigation for Subsurface Drainage

Subsurface drainage investigations involve most of the items pertinent to surface drainage plus more detailed information on soil, subsoil and ground water conditions. Ground water moves horizontally and vertically through the soil and subsurface materials. Therefore it is necessary to obtain the information on the permeabilities of these materials. Permeability vary with the type of soil or subsurface materials and with the structure and texture. Therefore it is necessary to investigate these parameters to the extent they have an influence on drainage.

The objectives of investigations for subsurface drainage are

- Detailed informations of drainage area
- Factors responsible for Groundwater excess or salinity
- Determination of the appropriate remedial measures
- Data collection for system design.

The various investigations usually required for subsurface drainage are:

i) Topographic Surveys
ii) Soil Investigation
iii) Irrigation
iv) Seepage
v) Hydrostatic Pressure
vi) Ground Water Studies
vii) Ground Water Contour Maps
viii) Depth to Water Table Maps
ix) Depth to Barrier Maps
x) Water Table Profiles
xi) Pirzometric Profile
xii) Hydrographs and
(xiii) Water Source Survey

(i) Topographic Surveys

In the case of subsurface drainage, topographic maps are needed to get information about the soils and ground water. High water table and seepage area must be marked on the topographic map. Drainage investigation in irrigated areas and deep areas of non-irrigated lands may require complete topographic survey. Normally the detail should be such that maps can be prepared with a contour interval in the range of 1 to 5 feet. The location and elevation of observation will or piezometers should be obtained along with other topographic data.

ii) Soil Investigation

For subsurface drainage, standard soil surveys are needed. Soils maps are useful. If available for drainage planning. Soil borings to approximately one and one-half times the estimated depth of drain are needed to determine depth and thickness of the different soil strata, location of layers of very low permeability and other soil property which should be considered in design. Layers may include clay pans, shale, sand stone, big iron, rock and gravel. If these layers exist, then design system may require inclusion of filters, gravel envelopes etc.

The pH of soil and amount of sulfates present must be known because they will have an effect on kinds of drain material that can be used.

(iii) Irrigation

The drainage problems in irrigated areas may be due to the application of too much irrigation water. To determine the extent to which irrigation practices causing such problems study should be made of (1) change in water table due to irrigation (2) water table fluctuations throughout the irrigation season and during times of no irrigation and (3) long term changes in water table elevation both before and after the beginning of irrigation. Deep percolation and surface runoff will be more if water is applied inefficiently. Irrigation method depends on various factors like soil, slope, crops, size of field and availability of water.

(iv) Seepage

Seepage is a major source of water in many drainage problem areas. Most seepage loss comes from canals carrying irrigation water, laterals reservoirs or the irrigation of higher lying lands. A comparison should be made of water level hydrographs with canals carrying full of water and when they are empty and with reservoirs to know the source of seepage water. Seepage can also be deleted by using radioisotopes, dyes, salts, observation holes and piezometers.

(v) Hydrostatic pressure

Hydrostatic pressure or artesian pressures are found where a slowly permeable layer overlies a saturated permeable layer that is under pressure. Water may be forced upward by the hydrostatic pressure through the slowly permeable layer. This excess water may be present in the areas where old artesian wells are seaking below the ground surface.

(vi) Ground water studies

The purpose of ground water study is to collect the informations on water table fluctuation and its position. The water table investigation also gives the quantity and direction of movement of ground water. To obtain this, it is necessary to have observation wells or piezometers, just outside and adjacent to the project area.

Water table elevation must be plotted against time on graph paper. The time scale which is usually on the abscissa is in days. The water table elevation is usually on the ordinate and is in cms or feet. By plotting the elevation value on the graph, it is possible to visualize the water table behaviour at that observation well. One full annual cycle of readings is needed before locating and designing a drainage system. In irrigated areas, water table levels fluctuate considerably during the growing season. Water table fluctuation may be in the order of 1.5m-2.5m. Under these situations, measurement should be done frequently to get the high and low water elevations. Measurement must be made 2-3 time per month for a period of several month to a year. By comparing monthly water table data, the highest water table condition can be known. This information will be extremely useful for designing drainage system.

vii) Ground water contour maps

The ground water contour maps must be prepared for finding the direction of water movement. The map can be prepared by measuring the water table elevation at selected points at a particular time and drawing the elevations on the base map of the project area. By interpolation, lines of equal water table elevation can be drawn on the map. These lines are termed as ground water contours and the map on which these contours were drawn is referred to as a ground water contour map.

viii) Depth to water table maps

To know the relationship between surface configuration and water table configuration, the ground water contour maps should be superimposed on topographic map. At any specific point on the map, the depth of water table is the difference in elevation between the surface contour and the ground water contour.

ix) Depth to Barrier maps

To know this, sufficient information on the barrier location must be available. A depth to barrier map can be prepared in a similar manner as depth to water table maps. This information is useful in making drain locations, estimating quantity of ground water movement etc.

x) Water table profiles

A water table profile can be made for a series of observation holes. The base profile is prepared by plotting the ground surface elevation, location and depth of observation holes and any ponds or canal in that profile. The elevation of the water surface at each observation hole can be plotted on the profile map. A water table profile is more useful it also contains information on subsurface material. The logs of observation hole can be plotted at respective hole. The elevation of the barrier in each hole should also be plotted on the profile. This will be helpful for locating drains in funding other drainage requirements.

xi) Piezometric profile

The piezometer is a useful tool in determining ground water conditions, particularly where water is under artesian pressure. In areas where artesian pressure are suspected piezometers should be installed in groups of 2 to 4 piezometers of different length. The number of piezometers depends on the variation in the subsoil profile. The elevation of the piezometric water table for each piezometer can be plotted at the elevation of the bottom of that piezometer located on the map. Equal water table elevations are joined by lines called equipotential lines. To determine the path of flow lines are drawn from higher elevation through lower elevation and perpendicular to the equipotential lines shows the direction of movement of water. The procedure is particularly useful in locating an artesian water sources.

xii) Hydrographs

Water table elevation must be plotted with respect to time for any observation well or piezometer to study the trend in water table movement. Rainfall, River stage, periods of canal operation can also be plotted on hydrograph to get more information regarding movement and pattern of water flow.

xiii) Water source survey

The principal influence of climate on drainage requirement depends on whether the cliamte is humid or arid. If the climate is humid, removal of excess surface and subsurface water originating from rainfall is the principal purpose for agricultural drainage. If the climate is arid, controlling the water table and preventing an accumulation of salt in the soils root zone resulting from irrigation water applications are the important factors. Factors related to climate the control the amount of water to be removed by drainage are the soil.surface condition, which controls the part of the rain falling that penetrates into the deep soil layers;

rainfall frequency; evapotranspiration and irrigation application in excess of evapotranspiration needs.

A survey should be made of the historic hydrology and climate trend in order to determine its relationship to water table fluctuations. The source of waters coming into the area must be determined. The water sources often governs the type of drainage to be installed. If excess water is due to precipitation, the remedial measure would probably be better surface drainage; if due to canal seepage, an interception drain may be indicated, and if due to artesian pressure; pumped wells provide the most practicable remedy. The common sources of water of major importance in drainage problem areas are precipitation irrigation and seepage.

4.2 FIELD INESTIGATION:

Several field methods have been proposed in literature for determining insitu permeability. These methods are described below.

4.2.1 Hvorslev's Point Piezometer Test:

It is possible to determine in situ hydraulic conductivity value by means of test carried out in single piezometer. Point piezometers are open only over a short interval interval at their base. Slotted piezometers are open over the entire thickness of a confined aquifer. Both tests are initiated by causing an instantaneous change in the level in piezometer through a sudden introduction or removal of a known volume of water. The recovery of the water level with time is then observed. When water is removed, the tests are often called bail tests; when it is added, they are known as slug tests. It is also possible to create the same effect by suddenly introducing or removing a solid cylinder of known volume.

The simplest interpretation of piezometer-recovery data is that of Hvorslev (1951). His initial analysis assumed a homogeneous, isotropic, infinite medium in which both soil and water are incompressible. With reference to the bail test of Fig. 4.1 (a), Hvorslev reasoned that the rate of inflow, q, at the piezometer top at any time t is proportional to the hydraulic conductivity, K, of the soil and to the unrecovered head difference, H-h, so that

$$q(t) = \pi r^2 \frac{dh}{dt} = FK\,(H-h) \qquad (4.2.1.1)$$

where F is a factor that depends on the shape and dimensions of the piezometer intake. If $q = q_0$ to $t = 0$, it is clear that q(t) will decrease asympotically toward zero as time goes on.

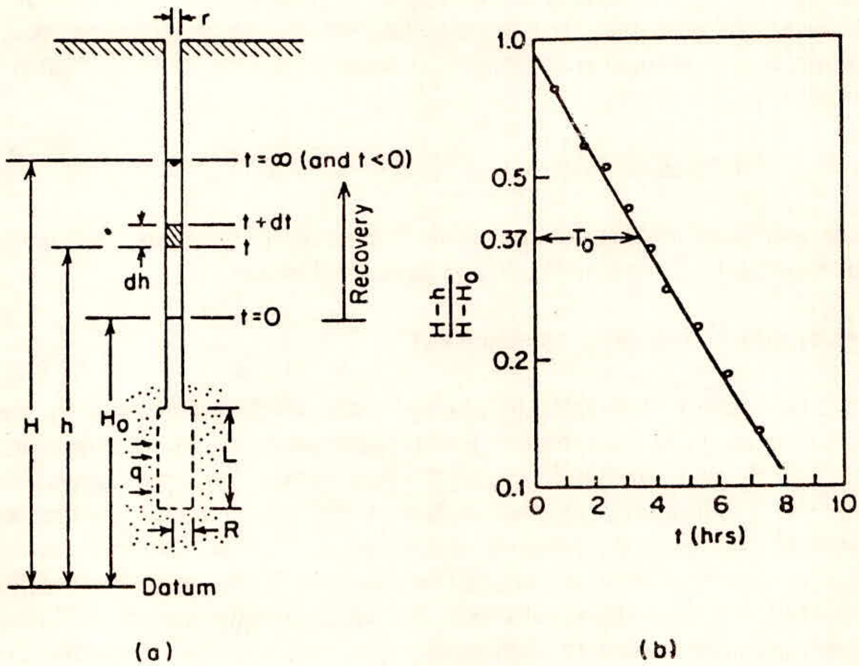


Figure 4.1. Horslev's Piezometer Test
(a) Geometry; (b) Method of Analysis

Hvorslev defined the basic time lag, T_0, as

$$T_0 = \frac{\pi r^2}{FK} \qquad (4.21.2)$$

When this parameter is substituted in Eq. (4.1), the solution to the resulting ordinary differential equation, with the initial condition, $h=H_0$ at $t = 0$, is:

$$\frac{H-h}{H-H_0} = e^{-t/T_0} \qquad (4.2.1.3)$$

A plot of field recovery data, H-h versus t, should therefore show an exponential decline in recovery rate with time. If, as shown on Fig. 4.2.(b), the recovery is normalized to H-H and plotted on a logarithmic scale, a straight-line plot results. It can be noted that for (H-h)/(H-H0) = 0.37, $\ln [(H-h)/(H-H_0)] = -1$, and from Eq. (4.3), $T_0 = t$. The basic time lag, T_0, can be defined by this relation or if a more physical definition is desired, it can be seen, by multiplying both top and bottom of Eq. (4.2.) by $H-H_0$, that T_0 is the time that would be required for the complete equalization of the head difference if the original rate of inflow were maintained. That is, $T_0 = V/q_0$, where V is the volume of water removed or added.

To interpret a set of field recovery data, the data are plotted in the form of Fig. 4.1(b). The value of T_0 is measured graphically, and K is determined from Eq. (4.2.). For a piezometer intake of length L and radius R with L/R >8, Hvorslev (1951) has evaluated the shape factor, F. The resulting expression for K is

$$K = \frac{\pi^2 \ln (L/R)}{2LT_0} \qquad (4.4)$$

4.2.2 Bail Test in Slotted Piezometer:

For bail tests or slug tests run in piezometers that are open over the entire thickness of a confined aquifer, Cooper et al. (1967) and Papadopoulos et al. (1973) have evolved a test-interpretation procedure. Their analysis if subject to the same assumptions as the solution for pumpage from a confined aquifer. Contrary to the Hvorslev method of analysis, it includes consideration of both formation and water compressibilities. It utilizes a curve-matching procedure to determine the aquifer coefficients T and s. The hydraulic conductivity K can then be determined on the basis of the relation, K = T/b.

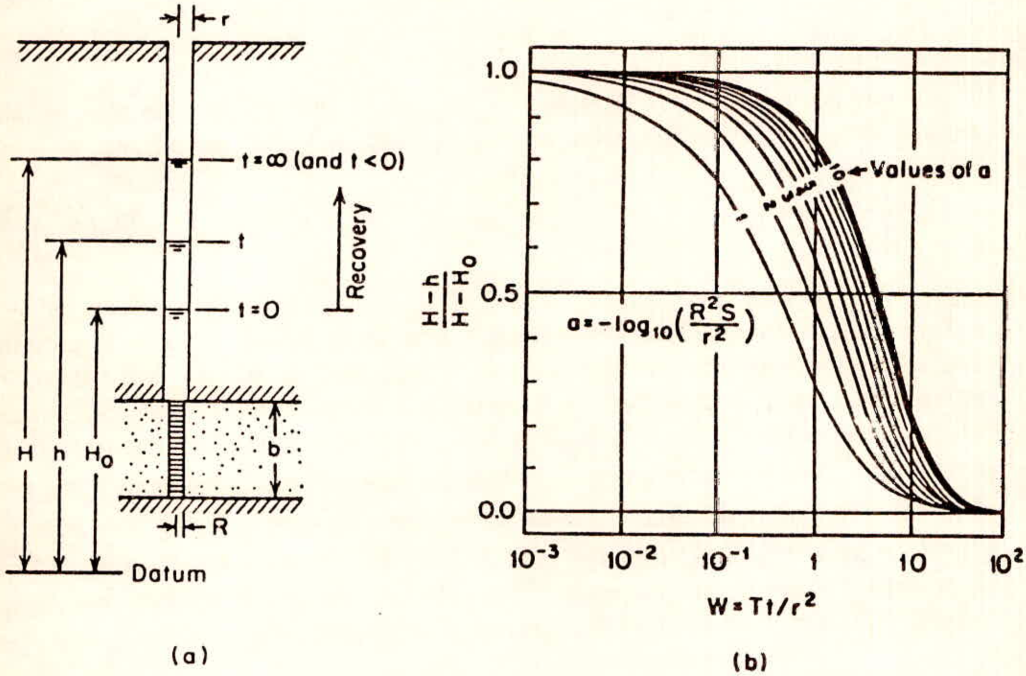


Figure.4.2: Piezometer Test in a Confined Aquifer
(a) Geometry;(b)Type Curves

For the bail-test geometry shown in Fig. [4.2(a)], the method involves the preparation of a plot of recovery data in the form $(H-h)/(H-H_0)$ versus t. The plot is prepared on a semilogarithmic paper, the $(H-h)/(H-H_0)$ scale is linear, while the ti scale is logarithmic. The field curve is then superimposed on the type curves shown in Fig. [4.2(b)]. With the axes coincident, the data plot is translated horizontally into a position where the data best fit one of the type curves. A matchpoint is chosen and values of t and W are read off the horizontal scales at the matched axis of the field plot and the type plot respectively. For ease of calculation it is common to choose a matched axis at W = 1.0. The transmissivity T is then given by

$$T = \frac{Wr^2}{t} \tag{4.5}$$

In principle, the storativity, s, can be determined from the value of the matched curve and the expression shown on Fig. 4.2.(b). In practice, since the slopes of the various lines are very similar, the determination of s by this method is unreliable.

The main limitation on slug tests and bail tests is that they are heavily dependent on a high-quality piezometer intake. If the wellpoint or screen is corroded or clogged, measured values may be highly inaccurate. On the other hand, if a piezometer is developed by surging or backwashing prior to testing, the measured values may reflect the increased conductivities in the artificially induced gravel pack around the intake.

4.2.3 Auger Hole Method:

The auger hole method can be used to measure hydraulic conductivity in situ below a water table. A detailed description of the procedure is given by Van Beers (1958). The principle of the method is as follows. A hole is bored into the soil with an auger to a certain depth below the water table. When the water in the hole reaches equilibrium with the groundwater, part of it is removed. The groundwater then begins to seep into the hole and rate at which it rises is measured. The hydraulic conductivity of the soil is computed by a formula or graph describing the mutual relation between the rate of rise, the groundwater conditions, and the geometry of the hole. This method measures the average hydraulic conductivity of a soil column about 30 cm in radius and extending from the groundwater table to about 20 cm below the bottom of the hole, or to a relatively impermeable layer if it occurs within 20 cm from the bottom of the hole.

Simple and convenient measuring equipment has been developed which consists of a tube, 60 cm long, the bottom end of which is fitted with a clack valve so that it can be used as a bailer. Extension pieces can be screwed to the top end

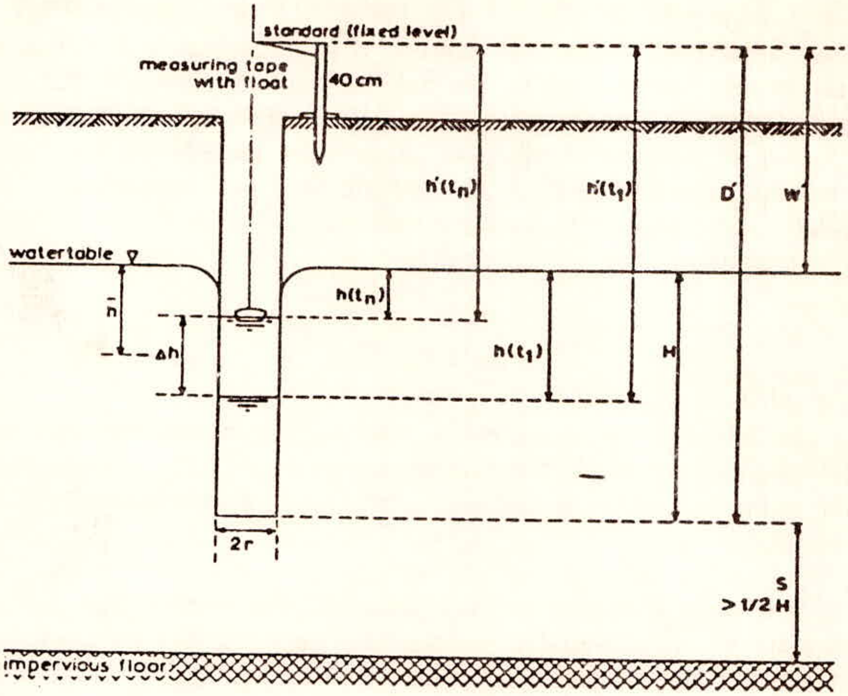


Fig. 4.3 The Auger Hole Method.

where

D'	= depth of the auger hole below level of the standard
W'	= depth of the water table below level of the standard
H	= D'-W', depth of the auger hole below water table
$h'(t_1)$, $h'(t_n)$	= depth of the water table in the hole below standard level at the time of the first reading(t_1)and after some reading (t_n)
Δh	= $h'(t_1)$-$h(t_n)$ the rise of water level in the hole during the time of the measurements
h	= $h(t_1)$- $0.5\Delta h$ = average head during the time of measurement
s	= depth of impervious floor below the bottom of the hole
r	= radius of the hole

of the tube. A float, a light-weight steel tape, and a standard are also part of the equipment. The standard is pressed into the soil up to a certain mark, so that the water level readings can be taken at a fixed height above the ground surface.

The hole must be made with a minimum of disturbance to the soil. The open blade auger is very suitable for wet clay soils, whereas the closed posthole auger is excellent in dry soils. The depth of the holes depends on the nature, thickness, and sequence of soil layers, and on the depth at which one wishes to determine hydraulic conductivity. When the water in the hole is in equilibrium with the groundwater, the level is recorded. Water is then bailed out to lower the level in the hole by 20 to 40 cm.

Measurement of the rate of rise in the water level must begin immediately after bailing. Either the time can be recorded. The first technique requires the use of chronometers while the second, which needs only a watch with a good second hand. Normally some 5 readings are taken, as these will give a reliable average value for the rate of rise and also provide a check against irregularities. The time interval at which water level readings are taken is usually from 5-30 seconds, depending on the hydraulic conductivity of the soil, and should correspond to a rise of about 1 cm in the water level. A good rule of thumb is that the rate of rise in mm/sec in an 8 cm diamteter hole to a depth of 70 cm below the water table approximately equals the K-value of the soil in m/day. Care should be taken to complete the measurements before 25% of the volume of water removed from the hole has been replaced by inflowing groundwater. After that a considerable funnel shaped watertable develops around the top of the hole. This increases resistance to the flow around and into the hole. This effect is not accounted for in the formulas or flow charts developed for the auger hole method and consequently it should be checked that $\Delta h < \frac{1}{4} h(t_1)$.

4.2.3.1 Computation of K by Ernst Formula:

For one layer soil Ernst (1950) found that the relation between the hydraulic conductivity of the soil and the flow of water into the auger hole depends on the boundary conditions. This relation has been derived numerically by the relaxation technique and is given as:

$$K = C \frac{\Delta h}{\Delta t} \qquad (4.2.3.1.1)$$

in which

K = hydraulic conductivity (m/day),

C = geometry factor = f(h,H,r,S) given in Fig. [4.4].

$\Delta h/\Delta t$ = rate of rise of water level in the auger hole (cm/sec). In Fig. [4] is given as a function of h/r and H/r for S > 0.5H. In Fig. [4.5] value of C is given for S = 0.

4.2.3.2 Computation of K by Hooghoudt's Method:

The Hooghoudt's method for computation K in the case where the auger hole does not reach to the impervious layer, is given by:

$$K = \frac{4000r^2}{h\,(H+20r)\,(2-h/H)} \; \frac{\Delta h}{\Delta t} \qquad (4.2.3.2.1)$$

in which, h is the mean depth to the water level in the hole, equal to $h(0)-0.5\Delta h$; $\Delta h/\Delta t$ is the mean rate of rise of the water level in the bore hole. In the formula K is expressed in metres/24 hours. All the other quantities are in cm or in sec.

If the bore hole reaches the impermeable layer K is computed from:

$$K = \frac{3600r^2}{h\,(H+10r)\,(2-h/H)} \; \frac{\Delta h}{\Delta t} \qquad (4.2.3.2.2)$$

The method may be applied only for the following conditions:

$3 > r > 7$ cm ; $S > 0.5H$; $y < 0.25\, y_0$; $20 > H < 200$ cm ; $y > 0.2H$.

For practical cases it must be noted that it is advisable to measure in bore holes of different depths when layers with different hydraulic conductivity have been observed.

4.2.3.3 Luthin and Kirkham Method for Computation of K:

Another method proposed by Luthin and Kirkham (1949) is the so-called piezometer method. A tube is driven into the soil below the water table with or without a cavity at the end of the tube as shown in Fig. [4.6(a)]. The soil is augered out of the tube. Then water is pumped out of the tube and the rate of rise is measured.

The hydraulic conductivity is computed from the equation:

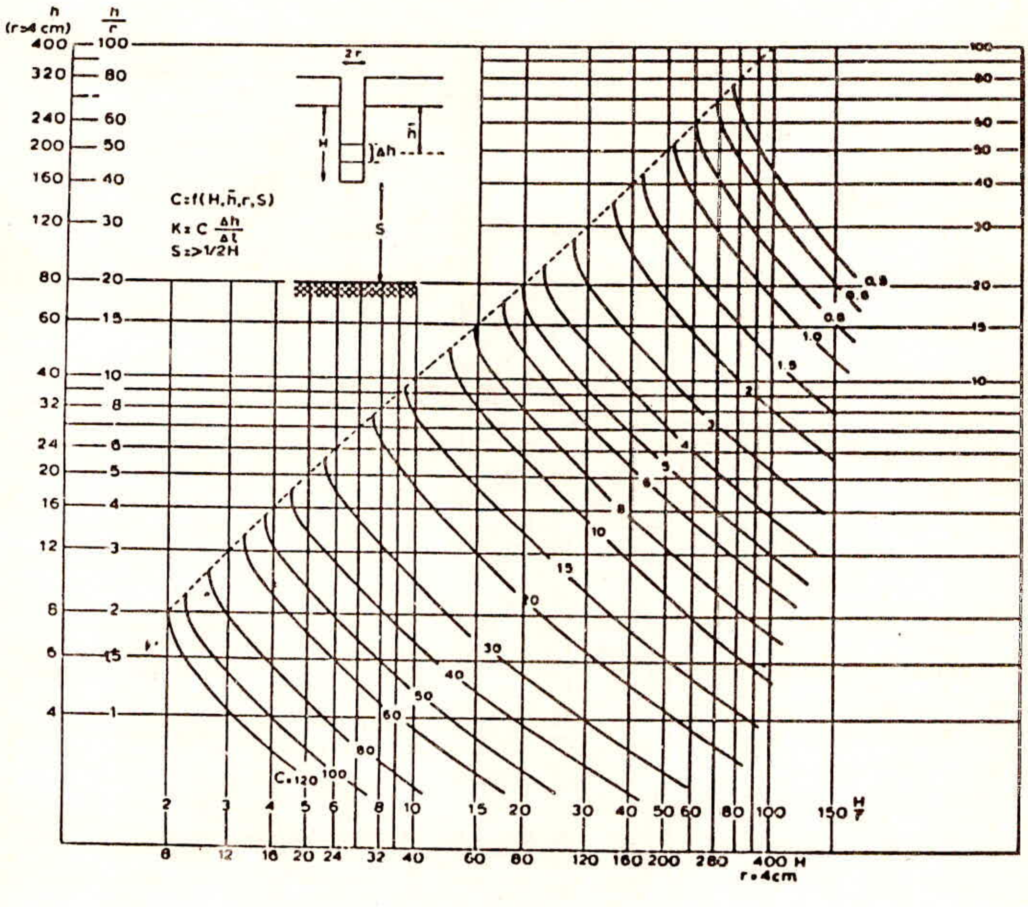


Fig.4.4 Nomograph for determination of C in auger-hole method for S >0.5H (Ernst,1950)

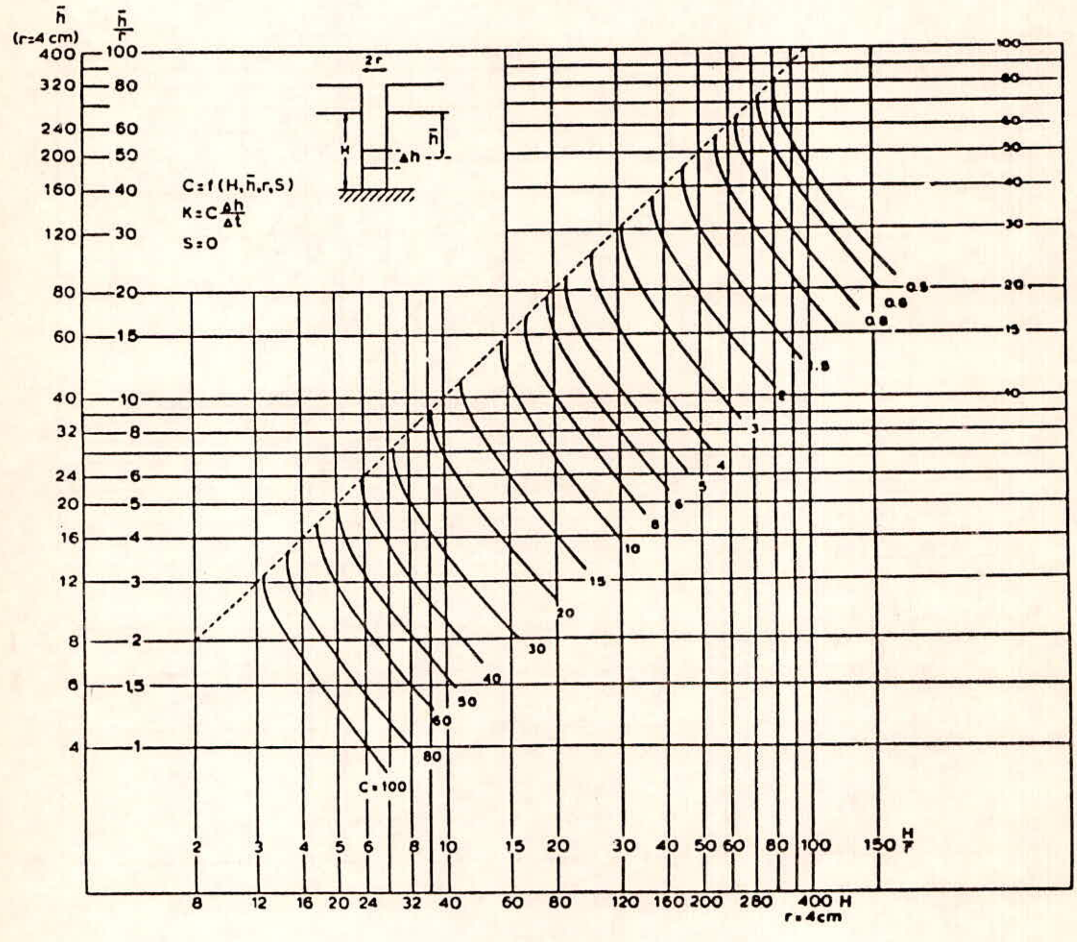


Fig.4.5: Nomograph for determination of C in auger-hole method for S=0 (Ernst,1950)

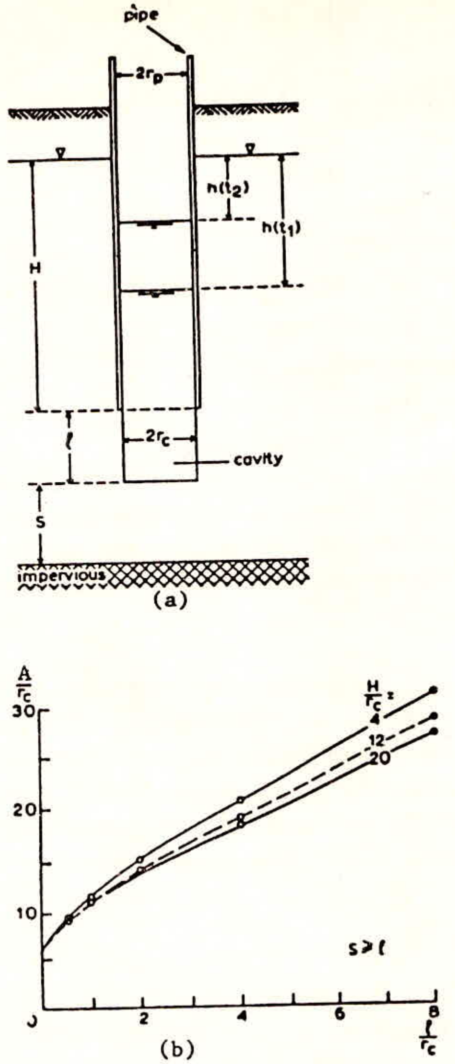


Fig.4.6: Computation of K by Luthin and Kirkham formula: (a) geometry (b) nomograph.

$$K = \frac{\pi r^2_p}{A(t_i - t_l)} \ln \frac{h(t_1)}{h(t_i)} \qquad (4.2.3.3.1)$$

in which

K	= hydraulic conductivity (cm/sec),
r_p	= inside radius of the pipe (cm),
$h(t_1), h(t_i)$	= depth of water level in cm in the pipe below the equilibrium water level at time t_1 and t_i respectively,
$t_i - t_1$	= time interval of measurement(sec), and
A	= $f(H, rc, l, s)$, geometry factor.

The A-function has been determined by means of an electric analogue and are given in Fig. [4.6(b)]. The piezometer method is very suitable for determining the hydraulic conductivity of individual soil layers. The separate layers, for which the hydraulic conductivity are to be determined, must have a minimum thickness of about twice the length of the cavity.

Both the auger hole and the piezometer method are conveniently applicable to a depth of 2 to 2.5 metres. If, especially for the purpose of determining possible deep groundwater flow or for the purpose of installing wells, the hydraulic conductivity must be known for larger depth, then a pumping test must be carried out. This means that the constants must be computed from the observed drawdowns around a well. The auger hole method can best be explained by means of an example.

Example of calculation:

An auger hole test is carried out with an auger of 8 cm diameter (r = 4 cm). A hole with a depth of 2 metres has been bored. The original depth of the water table was 74 cm below the ground surface. After pumping, the water table was at a depth of 105.2 cm. No impermeable layer within a depth of 3 metres was observed. Observation of the rise have been every 10 seconds. They are given in the table below.

From the above data

depth of bore hole		= 200 cm
depth to equilibrium watertable		= 74 cm
depth of bore hole below water table	H	= 126 cm

The observed water levels in the bore hole allow the following computations:

t(sec)	h(t)	$\Delta h(t)$	
0	105.2		$\Delta h(0)$ = 105.2-2.74=31.2 cm
10	104.0	1.2	Δh=h(0)-h(n)=5.6 cm
20	102.8	1.2	
30	101.7	1.1	h=h(0)-0.5Δ h=31.2-2.8=28.4 cm
40	100.6	1.1	
50	99.6	1.0	$\Delta h/ t_\Delta$=5.6/50=0.11 cm/sec
	Δh = 5.6		

Substituting h = 28.4 *cm*, r=4.0 *cm*, $\Delta h/\Delta t$ = 0.11 *cm/sec*, H=126 *cm in Eq.* (4.9) the value of K is found to be 0.67m/day.

4.2.4 Measurement of Field Saturated Hydraulic Conductivity by Guelph Permeameter

The auger hole method is a simple reliable technique for measuring saturated hydraulic conductivity in relatively uniform soils below the water table. However this method cannot be used if the water table is not present in the region of interest. *The methods for measuring hydraulic conductivity in the absence of* the water table are more complicated. The shallow well permeameter method, also known as the dry auger hole method and the bore hole permeameter method are used for measuring the hydraulic conductivity.

Hydraulic conductivity decreases as the soil water suction increases. This *relationship is called the conductivity pressure head relatioonship.* The Guelph permeameter is used to determine saturated hydraulic conductivity for a particular soil. Once the soil water suction is measured, the hydraulic conductivity for that soil at the soil water suction can be readily estimated.

4.2.4.1 Guelph Permeameter Apparatus

The Guelph Permeameter is essentially an 'in hole' Mariotte bottle constructed of concentric transparent plastic tubes. The apparatus comprises the following components as shown in Fig. 4.7.

i) Tripod Assembly
ii) Support Tubes and lower air tube fittings
iii) Reservoir Assembly
iv) Well Head Scale and upper air tube fittings
v) Auxiliary Tools

4.2.4.2 Theory

The Guelph permeameter method measures the steady state liquid recharge Q, necessary to maintain a constant depth of liquid H in an uncased cylindrical well of radius 'a', finished above the water table. Constant head level in the well hole is established and maintained by regulating the level of the bottom of the air tube which is located in the centre of the permeameter. As the water level in the reservoir falls, a vacuum is created in the air space above water. When the permeameter is operating, an equilibrium is established.

When a constant well height of water is established in a bored hole in a soil, a bulb of saturated soil with specific dimension is rather quickly established as shown in Fig. 4.8. The bulb is very stable and its shape depends on the type of soil, the radius of the well and the head of water in the well. The shape of the bulb is numerically described by the C factor used in the calculations. Once the bulb shape is established, the outflow of water from the well reaches a steady state flow rate which can be measured. The rate of this constant outflow of water, together with the diameter of the well and height of water in the well can be used to determine the field saturated hydraulic conductivity of the soil.

The Richard analysis of steady state discharge from a cylindrical well in unsaturated soil, as measured by the Guelph permeameter technique accounts for all the forces that contribute to three dimensional flow of water into soils; the hydraulic push of water into soil, the gravitational pull of liquid out through the bottom of the well and the capillary pull of water out of the well into the surrounding soil. The Richard analysis is the basis for the calculation of field saturated hydraulic conductivity. The C factor is a numerically derived shape factor which is dependent on the well radius 'a' and head 'H' of water in the well. Fig. 4.9 shows the curves for three classes of soil.

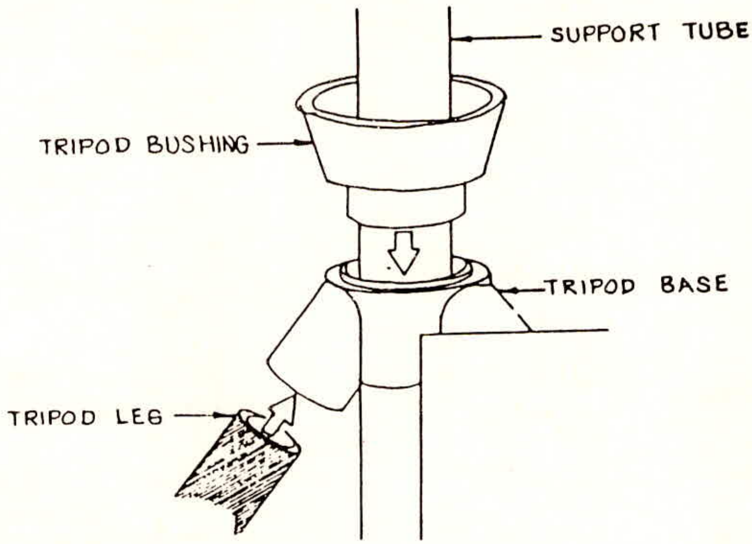


FIG 4.7 (a) TRIPOD ASSEMBLY OF GUELPH PERMEAMETER

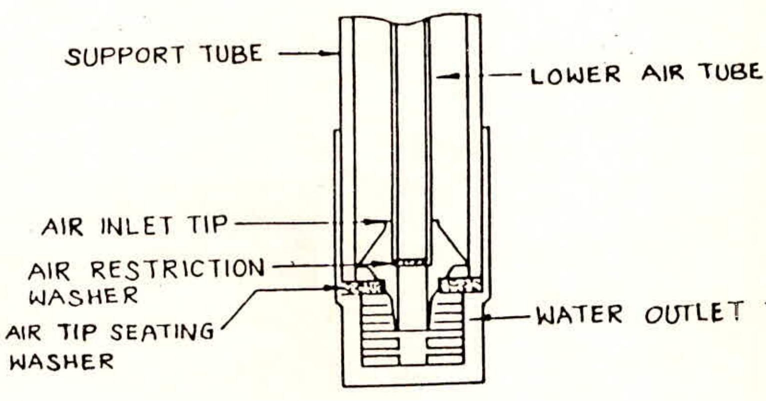


FIG 4.7(b) SUPPORT TUBE AND LOWER AIR TUBE FITTINGS OF GUELPH PERMEAMETER

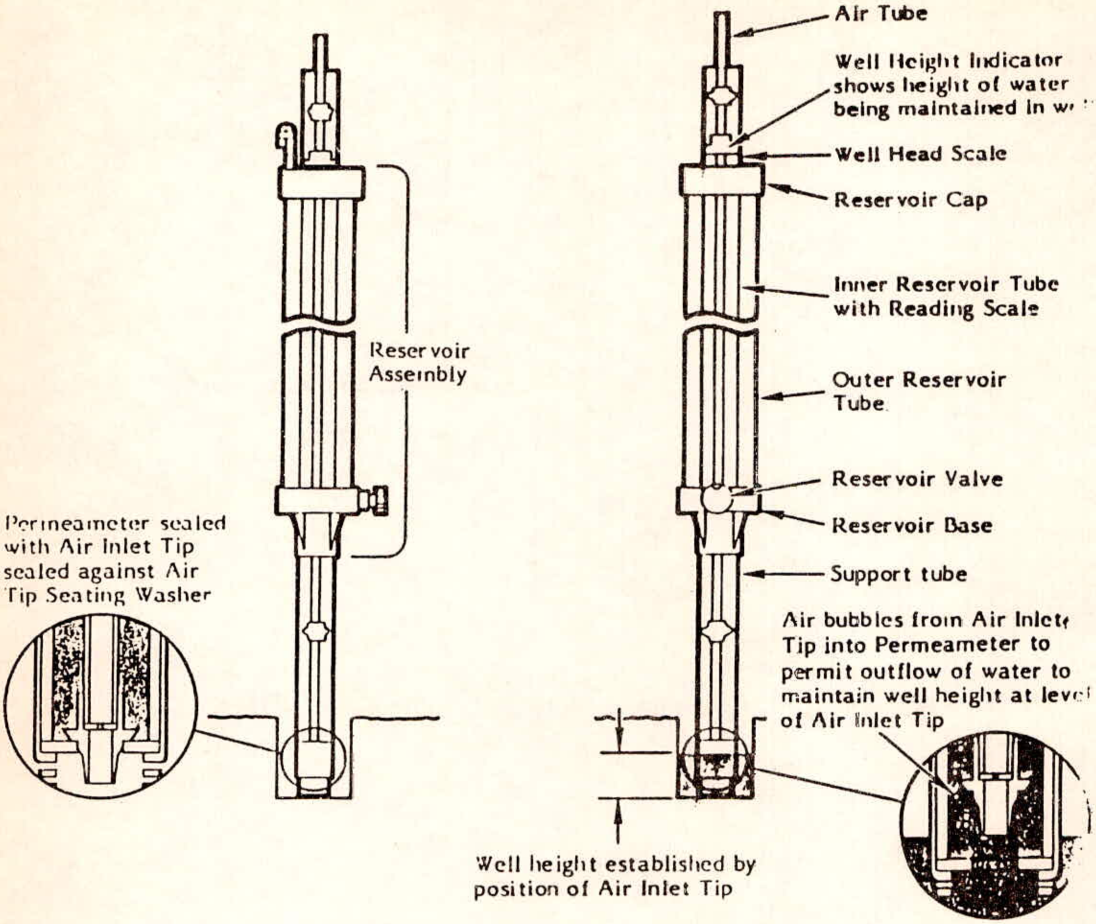


FIG. 4·7 (c) RESERVOIR ASSEMBLY

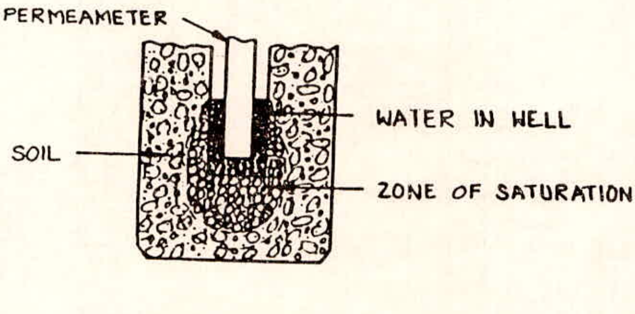


FIG. 4·8 A BULB OF SATURATED SOIL

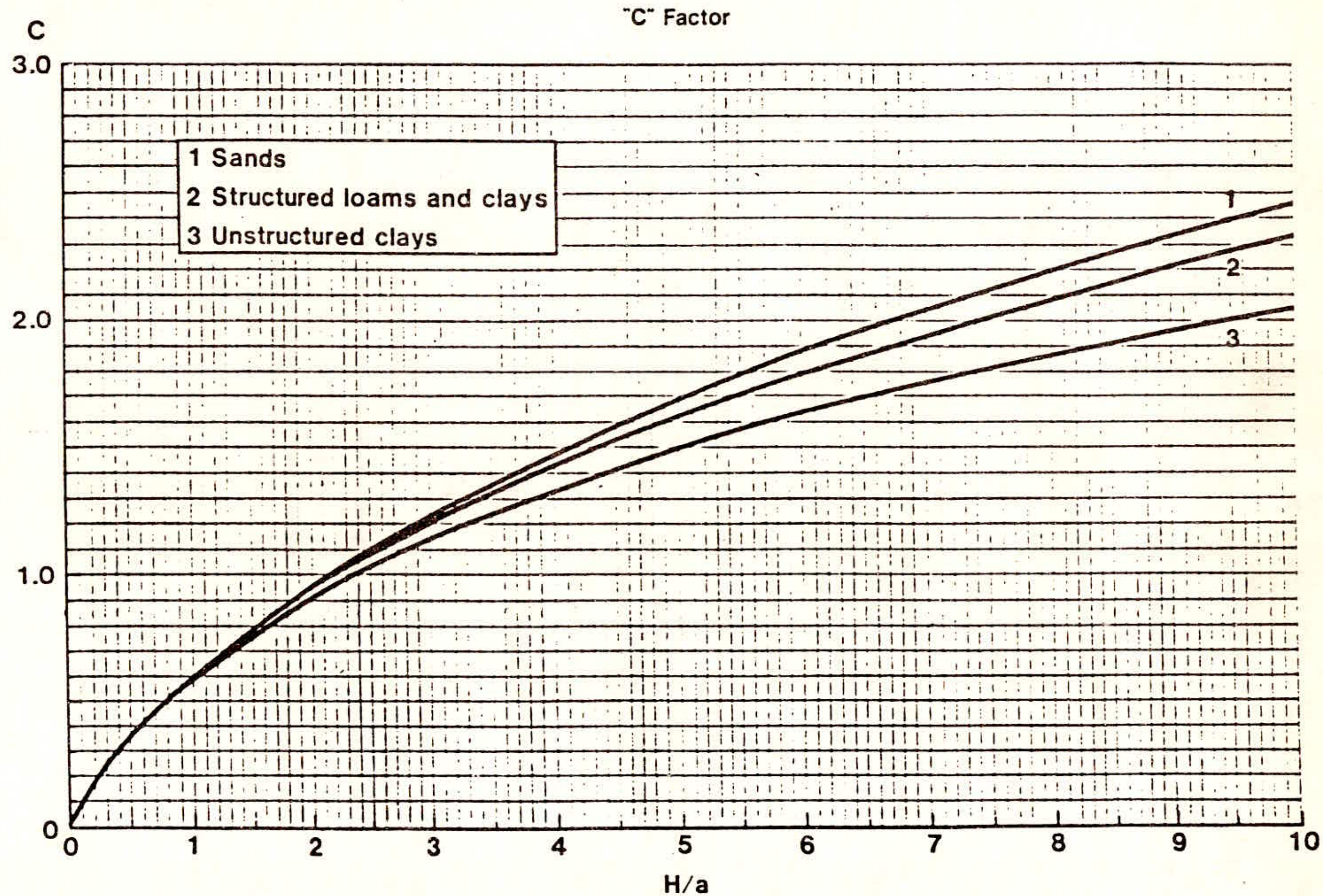


FIG.4.9- CURVES FOR THREE CLASSES OF SOIL

4.2.4.3 Procedures of Field Use

Before making a measurement with the Guelph permeameter in the field, it is necessary to perform a site and soil evaluation prepare a well hole, assemble the permeameter, fill the reservoirs, and place the permeameter in the well hole.

i) Well preparation :

The instruments needed for excavating and preparing a well bore hole are soil auger and sizing auger. The soil auger is used to remove bulk amounts of soil and rock. The sizing auger is used as a finishing tool to produce a proper sized well hole of uniform geometry and to clean debris off the bottom of the well hole. The sizing auger is designed to produce a hole that is uniformly 6 cm in diameter with a flat bottom. Generally, the procedure is to use the soil auger to excavate the well hole down to a depth 15 cm less than that desired for the final well hole. The last 15 cm can then be excavated using the sizing auger to produce a debris free well hole of uniform geometry.

In moist soils or in medium to fine textured soils, the process of augering a hole may create a smear layer which can block the natural flow of water out of the well into the surrounding soil. In order to obtain reliable and representative results using the Guelph Permeameter, the smear layer must be removed. The well prep brush is designed to use in the standard 6 cm diameter well hole.

ii) Permeameter Placement:

Simply centre the Tripod over the well hole and slowly lower the permeameter so that the support tube enters into the well hole. The tripod is used to support the permeameter in well down to approximately 38 cm in depth. For use in wells deeper than 38 cm, the tripod bushing alone provides the functions of centering and stabilizing the permeameter as shown in Fig. 4.10. After the permeameter is placed, it can be easily filled with water. The following standard procedure should be followed for making measurements.

a) Verify that both the reservoirs are connected. The reservoirs are connected when the notch on the reservoir valve is pointing up.

b) Established 5 cm Well Head Height (H1). Slowly raise the air inlet tip to establish the 5 cm well head height. Raising the air tube too quickly can cause turbulence and erosion in the well.

c) Observe the rate of fall of the water level in the reservoir. If it is too slow, then turn the reservoir valve so that the notch is pointing down. Water will then be

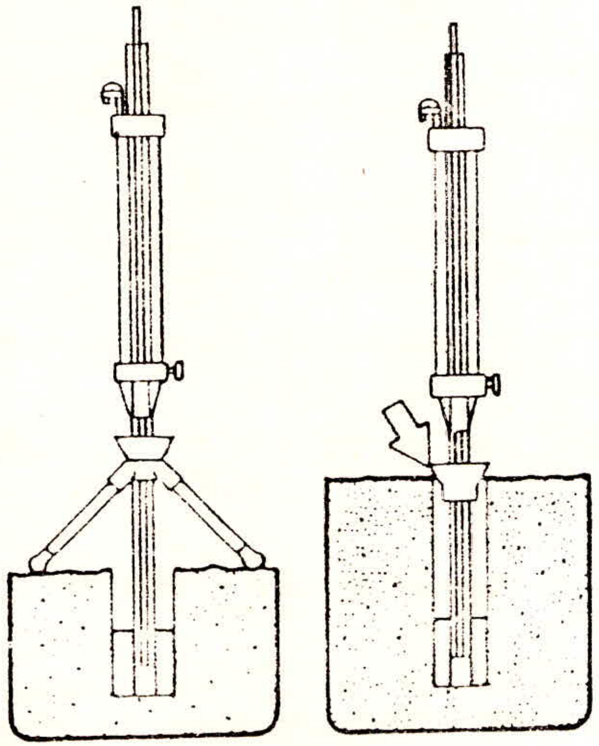


FIG.4.10 PERMEAMTER PLACEMENT

supplied, oniy from the small diameter inner reservoir which will result in a much greater drop in water level between readings.

d) Measure permeameter outflow. This is indicated by the rate of fall of water in the reservoir. Readings should be made at regular time intervals, usually 2 minute intervals are used. The difference of readings at consecutive interval divided by the time interval equals the rate of fall of water, R in the reservoir. Continue monitoring the rate of fall of water in the reservoir until the rate of fall does not significantly change in three consecutive time intervals. This rate is called R1 and is defined as the "Steady state rate of fall" of water in the reservoir at height H1 which is the first well height established and is always 5 cm in the standardized procedure.

e) Establish 10 cm Well head height (H2). Slowly raise the air inlet tip to establish the second well head height of 10 cm. Monitor the rate of fall of water, R, in the reservoir until a stable value of R is measured.

f) The field saturated hydraulic conductivity Kfs can be calculated using the following equation:

$$Kfs = 0.0041 \; .x. \; R2 - 0.005 \; .x. \; R1$$

where

x = Reservoir constant, equal to 35.39 when reservoir combination is used.

R2 = Steady, rate of fall of water in the reservoir when second head H2 equal to 10 cm of water is established,

R1 = Steady rate of fall of water in the reservoir when the first hed H1 equal to 5 cm of water is established,

Kfs = field saturated by canal in cm/sec.

4.2.4.4 Example

The experiment were carried out in the NIH campus to determine the field saturated hyd. cond. using Guelph permeameter. With the help of soil auger and sizing auger, a well hole of depth 43.0 cm was prepared. The first sets of reading with 5 cm head and second sets of reading with 10 cm head were taken and are shown in Table 4.10. The constant flow rate of 0.60 cm/min were observed after 14 minutes, when head is set at 5 cm. The constant flow rate of 1.50 cm/min were observed after 16 minute, when head is set to 10 cm.

Table 4.1 : Measurement of field saturated hydraulic conductivity ; standard procedure for permeameter readings

Reservoir constraints : Combined reservoir X=35.19 cm^2 ; Depth of well hole=43cm at NIH

Reading Number	Time (min)	Time interval (min)	Water level in reservoir. 5 cm	Water level change cm	Rate of water level change R1, cm/min
1	0	-	9.0	-	-
2	2	2	10.4	1.4	0.70
3	4	2	11.6	1.2	0.60
4	6	2	12.9	1.3	0.65
5	8	2	14.1	1.2	0.60
6	10	2	15.3	1.2	0.60
7	12	2	16.5	1.2	0.60
8	14	2	17.7	1.2	0.60

Reading Number	Time (min)	Time interval (min)	Water level in reservoir. 10cm	Water level change cm	Rate of water level change R1 cm/min
1	0	-	21.2	-	-
2	2	2	24.8	3.6	1.80
3	4	2	28.1	3.3	1.65
4	6	2	31.1	3.0	1.50
5	8	2	34.1	3.0	1.50
6	10	2	37.7	3.6	1.80
7	12	2	40.7	3.0	1.50
8	14	2	43.7	3.0	1.50
9	16	2	46.7	3.0	1.50

For the 1st set of Reading

$$R1 = 0.60/60 = .01 \text{ cm/sec., and}$$

For the 2nd set of Reading

$$R2 = 1.50/60 = 0.025 \text{ cm/sec.}$$

hence

$$Kfs = [(.0041)\ (35.19)\ (.025)] - [(.0054)\ (35.19)\ (.01)]$$

$$= 1.71 \times 10 \text{ Cm/Sec, or}$$

$$= 6.156 \text{ Cm/hour.}$$

4.2.5 Measurement of K by Dilution Technique:

It is also possible to determine hydraulic conductivity in a piezometer or single well by the introduction of a tracer into the well bore. The tracer concentration decreases with time under the influence of the natural hydraulic gradient that exists in the vicinity of the well. This approach is known as the borehole dilution.

The most direct method for groundwater velocity determination consists of introducing a tracer at one point in the flow field and observing its arrival at other points. After making adjustments for the effect of dispersion, the groundwater velocity can be computed from the travel time and distance data. Many types of nonradioactive and radioactive tracers have been used, ranging from such simple tracers as salt (NaCl or $CaCl_2$), which can be conveniently monitored by measurements of electrical conductance, to radioisotopes such as ^{3}H, ^{131}I, ^{29}Br, and ^{51}Cr- EDTA (an organic complex with ^{51}Cr), which can be accurately monitored using radioactivity detectors. Radioisotopes have the disadvantage of government licensing requirements for their use and of being hazardous when used by careless workers. Fluorescent dyes (florescein and rhodamine compounds) have been used by many investigators. In field tests, visual detection of the dye can sometimes yield adequate results. Dye concentrations can be measured quantitatively to very low concentrations when necessary. Recent work suggests that Freon (Cl_3CF) may be one of the best of the artificial tracers for use in groundwater velocity tests (Thompson et al., 1974). It is nonreactive with geologic materials and can be used in extremely small concentrations that are nonhazardous in public waters.

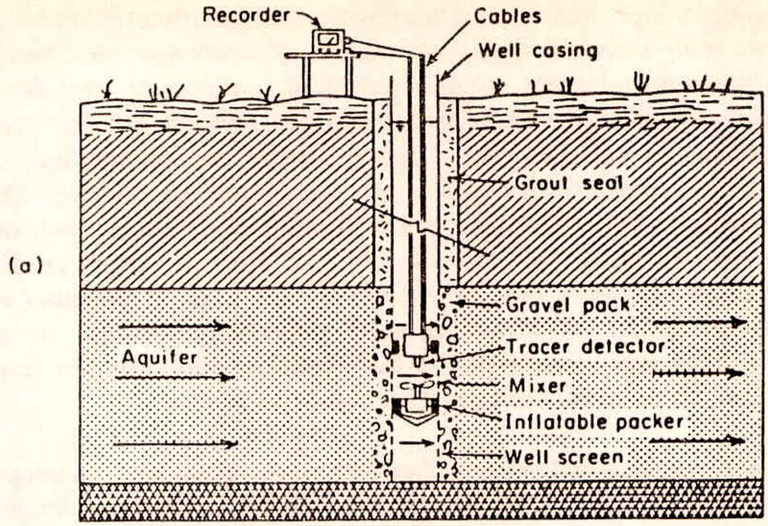
The direct tracer method of groundwater velocity determination described above has four main disadvantages: (1) because groundwater velocities are rarely large under natural conditions, undesirably long periods of time are normally required for tracers to move significant distances through the flow system; (2) because geological materials are typically quite heterogeneous, numerous observation points (piezometers, wells, or other sampling devices) are usually required to adequately the passage of the tracer through the portion of the flow field under investigation; (3) because of (1), only a small and possibly nonrepresentative sample of the flow field is tested; and (4) because of (2), the flow field may be significantly distorted by the measuring devices. As a result of these four factors, tracer experiments of this type commonly require considerable effort over extended periods of time and are rarely performed.

A tracer technique that avoids these disadvantages was developed in the USSR in the late 1940's. This technique, which has become known as the borehole dilution or point-dilution method, is now used extensively in Europe. Borehole dilution tests can be performed in relatively short periods of time in a single well or piezometer. The test provides an estimate to the horizontal average linear velocity of the groundwater in the formation near the well screen. A schematic representation of a borehole dilution test is shown in Fig.4.12(a). The test is performed in a segment of a well screen that is isolated by packers from overlying and underlying portions of the well. Into this isolated well segment a tracer is quickly introduced and is then subjected to continual mixing as lateral groundwater flow gradually removes the tracer from the well bore. The combined effect of groundwater through-flow and mixing within the isolated well segment produces a dilution versus time relation as illustrated in Fig.4.12(b). From this relation the average horizontal velocity of groundwater in the formation beyond the sand or gravel pack but close to the well screen is computed. The theory on which the computational methods are based is described below.

The effect of the well bore and sand pack in a lateral flow regime is shown in Fig. 4.13. The average linear velocity of the groundwater in the formation beyond the zone of disturbance is v. The average bulk velocity across the center of the well bore is denoted by v^*. It will be assumed that the tracer is nonreactive and that it is introduced instantaneously at concentration C_o into the isolated segment of the well screen. The vertical cross-sectional area through the center of the isolated segment is denoted as A. The volume of this well segment is W. At time t>0, the concentration C in the well decreases at a rate

$$\frac{dC}{dt} = -\frac{A \cdot v^* \cdot C}{W} \tag{4.2.4.1}$$

which, upon rearrangement, yields



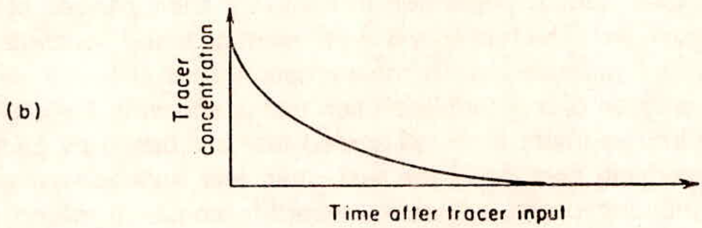


Figure 4.12 Borehole Dilution Test:

(a) Schematic Digram of Apparatus;

(b) Dilution of Tracer with Time.

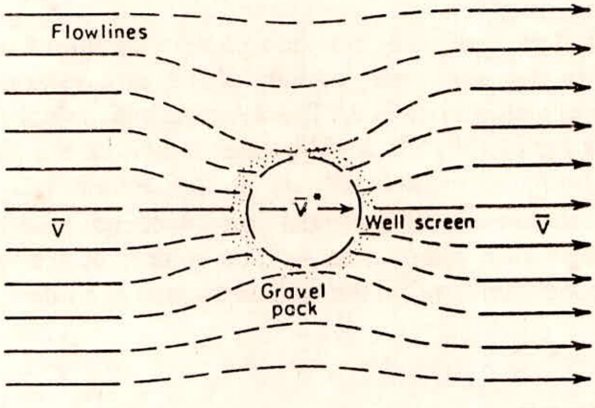


Figure 4.13 Distortion of Flow Pattern Caused by the Presence of the Well Screen and Sand or Gravel Pack.

$$\frac{dC}{C} = - \frac{A \, . v^* . dt}{W} \qquad (4.2.4.2)$$

Integration and use of the initial condition, C = C at 0, leads to

$$v^* = - \frac{W}{A \, . \, t} \ln (C/C_O) \qquad (4.2.4.3)$$

Thus, from concentration versus time data obtained during bore hole dilution tests values of v^* can be computed.

The objective of the test, however, is to obtain estimates of v. This is accomplished using the relation

$$v = \frac{v^*}{\alpha} \qquad (4.2.4.4)$$

where α is an adjustment factor that depends on the geometry of the well screen, and on the radius and hydraulic conductivity of the sand or gravel pack around the screen. The usual range of α for tests in sand or gravel aquifer is from .5 to 4 (Drost etal., 1968).

The borehole dilution method is described in detail by Halevy et al. (1968). In most borehole dilution tests described in the literature, radioactive tracers were used. The recent advent of commercially available electrodes for use with portable pH meters for rapid down-hole measurement of Cl or F has made it feasible to conduct borehole-dilution tests with these readily available tracers in a more convenient manner than was previously the case. An even simpler approach involves the use of salt as the tracer with down-hole measurement of electrical conductance as the salt is flushed from the well screen. Borehole dilution tests, like many other types of field tests used in groundwater studies, can be accomplished using simple, inexpensive equipment or more elaborate instrumentation. The choice of method depends on factors such as the hydrogeologic setting, availability of instrumentation, and the experimental precision and reproducibility that is desired.

Once v is computed, the hydraulic conductivity K can be determined by knowing the gradient of the water table in the vicinity of the bore hole.

4.3 LABORATORY METHODS

Laboratory measurements of hydraulic conductivity are conducted on soil samples contained in cylinders of known dimensions. If the hydraulic conductivity values are to be representative of a soil in situ, undisturbed samples must be obtained. Stainless steel cylinders with a thin wall and one sharpened end are used to extract soil samples above the groundwater table (Kopecky rings of 50 cm diameter x 51 mm length, wall thickness 1.5 mm). They are pressed gradually and evenly into the face of a profile pit. Care should be exercised to minimize soil compaction. The soil around the cylinder is then removed and the cylinder containing the sample is withdrawn. The ends of the sample should not be cut with a knife but should be removed to expose the natural structure of the soil. If the profile pit is cut in steps, vertical samples can also be taken. Undisturbed samples from the zone under the groundwater table can be obtained by using a coring apparatus and driven into the soil at the bottom of the bore hole. Closing off the tube above and below the sample with inflatable rubber rings prevents the loss of material during extraction. These samples can only be taken in vertical direction.

Two general laboratory methods are available for determining the coefficient of permeability of a soil directly. These are: the constant-head method, and the falling-head method. Both methods use the Darcy's law.

4.3.1 Constant Head Method:

The hydraulic head or the total head which causes flow in a constant head permeameter remains constant during the experiment. A line diagram of a constant head permeameter is shown in Fig. 4.14.

The quantity of water flowing through a soil sample of known area and length in a given time are measured. Head water and tail water levels are kept constant by overflows. The tail water is caught in a graduate and the rate of discharge is measured. From Darcy's law.

$$K = q / (i A) \tag{4.3.1.1}$$

in which q = quantity of fluid flow in a unit time, K=coefficient of permeability (units of velocity), i = hydraulic gradient = h/L, h = differential head across the samples, L = sample length across which h is measured, and A = cross-sectional area of soil mass.

In highly impervious soils the quantity q is small, and accurate measurements of its value are not easily obtained. Therefore, the constant head permeameter is principally application to relatively pervious soils.

4.3.2 Variable Head Method:

Instruments operating under variable head furnish more accurate determinations of the permeability of impervious soils. The line diagram of a variable head permeameter is shown in Fig. 4.15. The essential characteristic of the method is that the quantity of percolating water is measured indirectly by observations on the rate of fall of the water level in the stand-pipe above the specimen. The length L and area A of the sample and the area a of the standpipe are measured. In addition observations are taken on at least two different levels of the water in the standpipe. The coefficient of permeability is given by:

$$K = \frac{2.3\, a\, L}{A\,(t_2 - t_1)} \log_{10} \frac{h(t_1)}{h(t_2)} \qquad (4.3.2.1)$$

where

K	= hydraulic conductivity (cm/day)
L	= length of the sample (cm)
$h(t_1)$, $h(t_2)$	= head at time t_1 and t_2 respectively (cm)
t_2-t_1	= time interval (days)
a, A	= cross-sectional area of standpipe and soil sample respectively (cm^2)

4.3.3 Limitation of Laboratory Method:

Neither the constant-head nor the falling-head laboratory method provides a reliable value for the coefficient of permeability of a soil. Reasons for this are varied, but the major ones are as follows:

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. Orientation of the in situ stratum to the flow of water is probably not duplicated. In sands, the ratio of horizontal permeability to vertical permeability may be 3 to 4 or more (k_h/k_v = 3 or more), and this generally always occurs. Even if the field void ratio is duplicated for sands, the relationship of k_h/k_v will probably be lost.

3. 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.

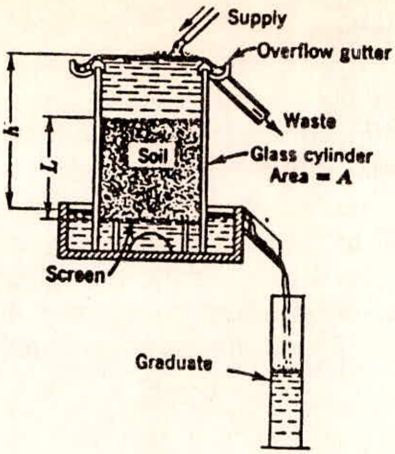


Figure 4.14 Constant head permeameter

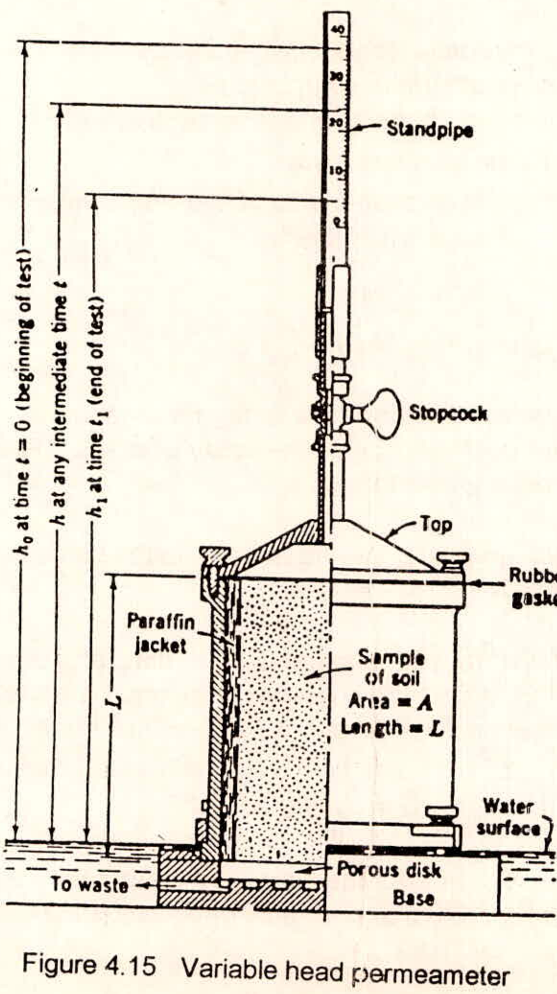


Figure 4.15 Variable head permeameter

If the soil is stratified vertically, the flow in the different strata will be different and this boundary condition may be impossible to duplicate.

4. The hydraulic head h may be different (often much larger) in the laboratory, causing a washout of fine material to the boundary, with a possible reduction of K. The field hydraulic gradient ($i=h/L$) is of the order of 0.5 to 1.5, indicates that $v = Ki$ is not linear for all values of i. 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 (Terzaghi (1925)].

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

4.3.4 Factors Controlling Permeability:

The coefficient of permeability of a homogeneous, isotropic soil mass depends primarily on the following factors:

i) The viscosity of the pore fluid (usually water)

As the temperature increases, i.e., the flow rate increases. The coefficient of permeability is standardized at 20 C, and the coefficient of permeability at any temperature T is related to K_{20} as in the following equation:

$$K_{20} = K_T \frac{\eta_T}{\eta_{20}} \tag{4.3.4.1}$$

where η_T and η_{20} are the viscosities of the fluid at the temperature T of the test and at 20 C, respectively. Either absolute or kinematic fluid viscosity may be used in Eq. (4.3.4.1). The values of absolute viscosity of distilled water at different temperature are given in the following table.

(ii) The degree of saturation:

As the degree of saturation increases, the apparent coefficient of permeability also increases. Part of this increase is due to the breaking of surface tension. The remainder is an unknown quantity, since it is difficult to determine k unless one considers the continuity of flow through the medium. The flow through the medium can be measured only by considering the quantity going into and coming out of the soil mass. As an extreme case one could, in a dry soil, have a considerable flow into the sample and no outflow at all. A computation of k would yield k=0, which is obviously incorrect. Saturated samples are generally used in the laboratory to avoid this problem, although recent investigation have been concerned with soil specimen which are not completed saturated.

Values of η for distilled water

Temp., ^0C	Viscosity of water, poise
4	0.01567
16	0.01111
17	0.01083
18	0.01056
19	0.01030
20	0.01005
21	0.00981
22	0.00958
23	0.00936
24	0.00914
25	0.00894
26	0.00874
27	0.00874
28	0.00836
29	0.00818
30	0.00801