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ON THE HYDROLOGY OF PEAT

by

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DECLARATION

I hereby declare that the following thesis is my own composition, that the work of which it is a record has been carried out be me, and that it has not previously been presented for a higher degree. I further declare that I was admitted by the Dean of the Faculty of Engineering and Applied Science as a research student in terms of Ordinance General No. 12 on 1st July, 1967.

DAVID WILLIAM RYCROFT.
1971.

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Abstract

The dissertation concerns hydrological work carried out on a raised peat bog in the North Eastern Highlands of Scotland. Subjects dealt with are the instrumentation of the catchment with water level recorders, instruments for recording evaporation and the associated meteorological parameters to act as a background to an investigation of methods for the estimation of the hydraulic conductivity of peat.

Methods used were the auger hold method (Kirkham and van Bavel 1943), the seepage tube method (Kirkham 1945) and the constant head permeameter method.

Initial results indicated that the apparent value of hydraulic conductivity varied in an unexpected manner. Further experiments confirmed the variations and showed that they occurred in response to variations of hydraulic gradient.

The manner in which the variations occurred was taken to indicate that they occurred in response to alterations of some property of the peat which might effect the manner in which Darcy's Law operates.

NOTATION

Each symbol is defined only when first encountered

A:	Shape factor	L
a:	Acceleration; radial dimension of a well	LT^{-2}; L
D;	Coefficient	
d:	Depth of well	L
F:	Dimension of force	MLT^{-2}
g:	Gravitational acceleration	LT^{-2}
H:	Height	L
I:	Electrical current (ampere)	$\mu^{-\frac{1}{2}}M^{\frac{1}{2}}L^{\frac{1}{2}}T^{-1}$
h:	Height: potential hydraulic head	L
K:	Hydraulic conductivity	LT^{-1}
K:	Intrinsic soil permeability	L^2
L:	Length	L
M:	Mass	M
N:	integer; constant of proportionality	-
n:	Porosity	-
P:	Hydraulic pressure	$ML^{-1}T^{-2}$
Q:	Discharge	L^3T^{-1}
q:	Specific flux	LT^{-1}
R:	Electrical resistance	μLT^{-1}
r:	Radius; radial distance in cylindrical co-ordinates	L
S:	Shape factor (dimensionless unless otherwise stated)	-
s:	Cross sectional area; co-ordinate distance on Z axis to impermeable barrier	L^2; L
t:	Time	T
u,v,w:	Velocity components in the x,y,z directions respectively	LT^{-1}
V:	Electrical potential (voltage)	$\mu^{\frac{1}{2}}M^{\frac{1}{2}}L^{3/2}T^{-2}$
v:	Velocity	LT^{-1}
W:	Length of surface of seepage below a seepage tube	L
x,y,z;	Cartesian co-ordinates (x usually horizontal; y horizontal in three dimensions, sometimes vertical in two dimensions)	
Z:	Elevation from a fixed datum	L

α, β	:	Transformation coefficients	-
Δ	:	Increment; difference	-
δ	:	Increment; difference	-
θ	:	Angle; phase difference	-
λ, ϵ	:	Transformed cartesian axes corresponding to y and z axes	-
μ	:	Dynamic viscosity	$ML^{-1}T^{-1}$
ρ	:	Mass density	ML^{-3}
Σ	:	Symbol for sum	-
σ	:	Electrical conductivity	$^{-1}L^{-2}T$
τ	:	Shear stress in the fluid	$ML^{-1}T^{-2}$
Φ		$K\phi$ = velocity potential in uniform soils	L^2T^{-2}
ϕ	:	$z + \frac{p}{\rho}$ = piezometric head or potential	L
Re^1	:	Reynolds number with respect to pore diameter	-

Subscripts

i, r, d	:	inertial; resistive; driving
i, j	:	Numbers
p	:	Porous
H	:	Horizontal
V	:	Vertical
x, y, z	:	Directional components

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SECTION 1.1

The Hydrological Behaviour of Peat Covered Catchments

1.1.(i) Introduction

Peat is an organic soil medium formed by the incomplete decomposition of plant residues. The resulting deposits are commonly described as mires.

The effect which mires exert upon the hydrology of an area is uncertain, and it is suggested that this is due to an ignorance of the fundamental processes occurring in them. The process with which this thesis is primarily concerned is the sub-surface seepage of water through peat.

1.1 (ii) Previous investigations into the hydrology of peat covered catchments

Conway and Millar (1960) investigated the hydrological behaviour of four small peat covered catchments on the upper reaches of the River Tees catchment area. They measured the total discharge of water from the catchments (the runoff) and the rainfall on them.

Two of the areas were traversed by artificial drainage ditches spaced at approximately twenty yard intervals, and one of them had, in addition, an extensive network of natural dendroid drains sharing a mutual outfall with the artificially drained section. The surface of this area had been burnt prior to the investigations. Of the two remaining catchments, one had a natural network of drainage ditches extending well into the mire, whilst the other was drained by one natural channel only. This area was considered to approximate to an active undisturbed mire.

It was found that the latter area could receive and store as much as 6" of rainfall. In contrast the intensively drained areas had very little capacity to store the rainfall. Thus river discharge from a drained area gave rise to an early and high peak flow. On the other hand, the stored water in the natural undrained peat covered catchment tended to be released more slowly, and in fact supported flow through dry periods. The burning had turned the surface of the mire concerned into an impervious cheesy cover which increased the rate of runoff.

Robertson, Nicholson and Hughes (1963) sought to measure the net input to and output of water (the water balance) from a 17 acre peat covered catchment area by measuring the rainfall on to the area, the evapotranspiration occurring from the vegetation covering the area, and the runoff resulting from the drainage of that area. They found that the evapotranspiration was highest between June and August, and that this affected the water table height which in turn affected the runoff from the area. When the water table was low the peat covered area could absorb and retain precipitation, indicating the importance of the antecedent hydrological conditions.

Baden and Eggelsmann (1964) worked on a peat covered catchment, the Königsmoor, in a low rainfall area of North West Germany, where the hydrological behaviour of natural undrained peat covered areas, and drained and cultivated peat covered areas was investigated. The results showed that the untouched area had little storage capacity whereas the drained area was able to store and release slowly that water which had fallen on it.

There is clearly a contradiction of conclusions between the work in Britain and Germany. This may be partly explained by Huikari (1959) who investigated the effect of the spacing and the depth of artificial parallel drainage ditches in peat on the resulting runoff. Ditches at 100 m. intervals had very little effect upon the natural water table in the peat, irrespective of their depth. Generally speaking a critical ditch spacing of 30 m. existed. In periods of high rainfall, ditches spaced closer together than 30 m. could receive up to four times as much runoff, at any one time, as those at 100 m. spacings. At close spacings the ditch depth became important. A 77 cm. depth drain was capable of sustaining flow throughout the season, whereas a ditch of only 46 cm. dried up as the water table fell below the ditch level. The pattern of the runoff/time relationship was considerably altered by the type of drainage treatment to which the mire had been subjected. The contradictory results previously quoted were obtained as a result of the measurement of discharge from mires at suitable outflows. The measured flow had drained from the mire by ditches or tile drains arranged in unique patterns, each of which would give rise to a unique runoff/time pattern. The results were considered to reflect mainly those physical properties of the mire which affected discharge, whereas in fact they reflected the combination of physical properties of the mire and the drainage system, evolved naturally or to which it had been subjected.

Bay (1966) worked on some small forested peat covered catchments in Northern Minnesota. He measured the runoff and some hydrophysical properties of the peat. Water levels were measured both in the mire and the surrounding mineral soil, and conclusions were drawn which paid attention to work on the basic physical properties of peat by Boelter (1964). The importance of the position of the water table, prior to precipitation, with respect to water storage was emphasised. It was pointed out that usually the heavy rain periods occurred at times when the available storage was negligible anyway. One of the mires was situated above the regional water table. The annual fluctuation in the water table level was larger in this, in contrast to the mire where the water levels in the mineral soil and the mire were continuous. The water seeping into the mire from the mineral soil maintained a high water table in that mire. Consequently it had less storage capacity. Bay later (1969) concluded that heavy rainfall was required in the summer months in order that a measurable runoff response should occur. Outside this period the mire was not effective as a stream regulator. Bay came to the conclusion that subsurface seepage, in that mire with a perched water table, did not contribute to stream flow and water received from the area was surface runoff. Graphs showing the relation between peak runoff and water table height were presented, and also a graph which reveals the delay intervening between the centroid of volume precipitation and peak flow. These curves are of course specific to the particular mires chosen.

Romanov (1968) suggests that the movement of water across a peat covered area is largely governed by the hydrophysical properties of the surface layer. This layer which contains the growing plants is defined as the active layer. Romanov drew three important conclusions concerning the hydrological investigations of mires. Firstly, that hydrometric measurements of discharge at terminal points provided little information on processes occurring in the mire massif itself and the data could not always be extended to other mire massifs. For example, the total runoff from a mire having areas supporting different plant associations would depend upon the size of these areas. The character of the runoff would, however, depend both upon the ratio of these areas and their relative dispositions with respect to the position at which the runoff was observed. Secondly, the hydrophysical characteristics of the upper active layer discouraged the use of conventional measurement methods, and thirdly, a knowledge of the relationship between the hydrological processes, i.e. runoff and evapotranspiration, and the environmental physical properties of peat, i.e. the energy exchange between the peat deposits and the atmosphere, was indispensible for applying data on runoff and evaporation to other climatic regions.

Considering these results, it is concluded that the apparent contradiction between the British and German findings is partly the result of the different treatments, i.e. drainage, to which the mires have been subjected.

Conway and Millar (1960) pointed out the difficulty of defining the boundaries of the catchments. This being so, it must also in general be true that a difficulty exists in separating the contributions to the observed river flow due to the peat covered area and to the remaining areas in the catchment.

The accurate interpretation of runoff characteristics requires a detailed knowledge of the fundamental factors which, in combination, produce the characteristics observed, and in general this information is not available.

SECTION 1.2

The Research Project

It was decided that a suitable peat covered area would be chosen and instrumented with a view to providing information on the general hydrology of the area which would act as a background to studies of a more specific nature.

Ideally measurements would be made of river discharge at a suitable outfall, of evapotranspiration from a suitable area, of the meteorological parameters affecting these relationships and of the response of the water levels in the mire to recharge or depletion.

The specific study to be undertaken by this author would be an assessment of methods for the determination of the hydraulic conductivity of peat, for it will be seen, although this is a most important factor in water movement determinations, its estimation in peat is itself in question.

SECTION 1.3

Hydraulic Conductivity

1.3 (i) The significance of hydraulic conductivity

Darcy (1856) experimentally derived a linear relationship between the rate of flow of water through a saturated bed of sand and the hydraulic potential which caused and maintained that flow. This may be written:

$$Q' = - K \frac{s \Delta \phi}{L} \qquad 1.3.1$$

Equation 1.3.1 applies to the flow of water through a column of porous material of length L and cross sectional area s. The volume flow rate Q' in the direction of L occurs in response to a hydraulic head difference $- \Delta \phi$. K is commonly called the hydraulic conductivity. If the flow velocity v is taken as $\frac{Q'}{s}$, and the hydraulic gradient i as $- \frac{\Delta \phi}{L}$ equation 1.3.1 may be re-arranged thus:

$$K = \frac{v}{i} \qquad 1.3.2$$

The value of hydraulic conductivity and its constancy depends upon the properties of the fluid which is being conducted and upon the structure and the nature of the porous medium.

The properties of water, relevant to this investigation and porous media are dealt with in the literature, see for example Childs (1969 chapters 3 and 4) and they will only be considered briefly here.

The water molecule is dipolar, and in the fluid state this results in a tetrahedral structure of four water molecules surrounding a central molecule. At any instant of time the tetrahedral structures in turn tend to produce interlocking spirals. The open molecular structure thus formed is responsible for many of the physical properties of the fluid. For example, density variations as a result of temperature changes are attributed to the changing molecular structure. The molecular structure is also thought to be modified by the presence of elements and compounds which are readily accepted by the fluid. Water molecules may interact with materials with which they are in contact. For example, they may be adsorbed onto the surface of a solid. A fluid layer of many molecules thickness may result and this layer could well exhibit different properties from the fluid unaffected by the surface.

For flow to occur through a porous medium there must be a series of interconnected pores. The interconnected porous structure is often characterised by the sizes of the individual pores. These may range from molecular interstices, where the voids are of the order of sizes of the molecules in the porous material and the fluid, to caverns, the size of which bear no relation to the smaller particles forming their walls. It is generally thought that the size of the pores in a medium are much larger than the connections between them through which the fluid must flow and which contribute largely to the resistance to flow observed. A porous medium will in general contain a proportion of pores which are isolated by the structure of the particles. These will be ineffective in conducting fluid.

The hydraulic conductivity may vary due to anisotropy or because the medium is not rigid but is structurally affected by the conduction of fluid. It is worth noting that peat is a very compressible organic medium and the water which is contained within the peat is recognised as being acidic.

1.3 (ii) Hydraulic conductivity and its relation to the theories of fluid motion

Childs (1969), Hubbert (1940) and Luthin (1957) give accounts of the original Darcy experiment, from which it is understood that Darcy experimented upon the flow of water through columns of sand, fig. 1.3.1 . He assumed that the volume of water Q flowing in time t down the column was directly proportional to the cross sectional area s normal to the direction of flow. By observing water flowing under various heads $(H_B + L - H_A)$ through columns of various lengths L he showed that the rate of flow was directly proportional to the head and inversely proportional to the length. Thus:

$$\frac{Q}{t} = K \frac{s}{L} (H_B + L - H_A) \qquad 1.3.3$$

$(H_B + L - H_A)$ is clearly the relative head of water causing motion, and may be described as the potential head referred to the outflow level. The hydraulic conductivity K has the units of velocity. A potential head may be described at any point in the fluid provided it is related to a particular datum. It is, however, the difference in potentials which causes the flow of water in the experiment above and indeed in all flow problems with which this study will be concerned. The potential difference between two points in a fluid is a direct expression of the amount of work required to raise a unit quantity of the fluid from the lower to the upper potential. The unit quantity may be volume, mass or weight and the units of potential are altered accordingly.

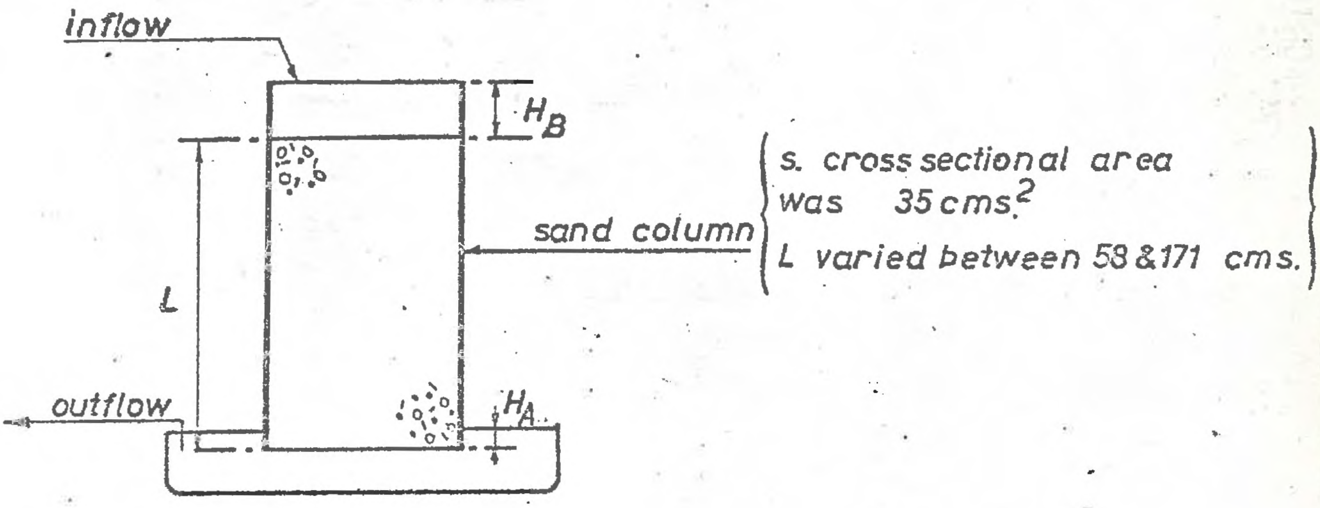
Darcy's equation may be re-written thus:

$$\frac{Q}{t} = Ks\,(\phi_B - \phi_A)/L \qquad 1.3.4$$

Let:

$$\frac{Q}{ts} = q$$

which is the flux through unit area in unit time,



The Experiment of Darcy

Fig (1.3.1)

and:

$$\phi_B - \phi_A = \Delta \phi$$

where ϕ, the potential, refers in the definition to the work expended upon unit **weight** of the fluid, then:

$$q = -K \frac{\Delta \phi}{L} \qquad 1.3.5$$

Darcy formulated his expression for rectilinear flow in a medium assumed to be isotropic, but a more general expression of his statement considers a small element of the flow system only in which in the limit of smallness, the potential gradient is constant. Thus:

$$v = q = -K \text{ grad } \phi \qquad 1.3.6$$

In this equation v is the velocity, a vector having magnitude and direction.

Further consideration of the properties of porous media has given rise to equations which allow for directional variations, i.e. the anisotropy of the conductivity. The variation is described in relation to mutually orthogonal axes x, y and z. In the introductory comments, it was pointed out that hydraulic conductivity was a function of the properties both of the fluid, and of the porous medium. Whilst this is clear from Darcy's expression, it is by no means clear which properties are important or how they may be related to the constant. Some indications follow from considering the development of concepts of fluid dynamics as they relate to porous media.

Newton's second law states that the total force acting on a particle is the product of its mass and acceleration, thus:

$$F = m a \qquad 1.3.7$$

The Eulerian approach to the problems posed by flowing water is adopted for the purpose of allowing the requisite equations to be developed. This considers the forces acting on fluid moving through a cube which is fixed in space. The position of the cube is described by the orthogonal axes x, y and z. Further consideration of Newton's second law led to the following expression for idealized fluids:

$$a_x = X - \frac{1}{\rho}\frac{\delta p}{\delta x} \qquad 1.3.8$$

This expression relates the acceleration, a_x, in the x direction, experienced by a unit mass of the fluid, to the forces which are acting in that direction to cause the acceleration. X is the component of a body force on unit mass, and δp is the pressure difference, across the length δx.

According to Richardson (1961), Euler proposed an equation which described the rate of change of a property of a fluid in a time dt which may, for instance, be the velocity. The expression states that the total change of the property du, which may be assumed to be velocity, with time dt is:

$$\frac{du}{dt} = \frac{\delta u}{\delta t} + u\frac{\delta u}{\delta x} + v\frac{\delta u}{\delta y} + w\frac{\delta u}{\delta z} \qquad 1.3.9$$

$\frac{\delta u}{\delta t}$ is the change which the property undergoes when the place of observation remains fixed, in other words, it is a local change. The remaining three terms, the convective changes, express the change of the property as the place of observation moves. By combining equations (1.3.8) and (1.3.9) Euler's equation is obtained:

$$a_x = \frac{du}{dt} = X - \frac{1}{\rho}\frac{\delta p}{\delta x} = \frac{\delta u}{\delta t} + u\frac{\delta u}{\delta x} + v\frac{\delta u}{\delta y} + w\frac{\delta u}{\delta z} \qquad 1.3.10$$

Similar expressions describe the accelerations a_y and a_z along the respective axes.

Navier (cit. Richardson 1961), and Stokes (1845) developed equations, similar in form, but applicable to real fluids, by including terms to allow for the viscosity of a real fluid, thus:

$$\frac{du}{dt} = X - \frac{1}{\rho}\frac{\delta p}{\delta x} + \mu dv\left(\frac{\delta^2 u}{\delta x^2} + \frac{\delta^2 u}{\delta y^2} + \frac{\delta^2 u}{\delta z^2}\right) \qquad 1.3.11$$

In this equation dv is the volume of the fluid element, ρ the mass density and μ the dynamic viscosity.

Hubbart (1940) presented the development of equations based upon these basic equations and necessary to the understanding of the variables in the hydraulic conductivity term.

For extremely slow flows the following relationship is true:

$$F_D = - F_R \qquad 1.3.12$$

F_D is the total driving force across a macroscopic element of the porous media in the direction of flow and F_R is the resulting resistance force. F_D is obtained by considering the pressure and gravitational forces acting on a cylindrical element of the porous medium of length δL and volume Δv. The following expression is derived:

$$F_D = \left(- g\frac{\delta z}{\delta L} - \frac{1}{\rho}\frac{\delta p}{\delta L}\right)\rho n \Delta v \qquad 1.3.13$$

where δp is the pressure difference across the length δL in a porous medium of porosity n. The expression within the brackets is the hydraulic potential gradient/unit mass and it therefore follows that:

$$\frac{F_D}{\rho n \Delta v} = - \frac{\delta \phi}{\delta L} \qquad 1.3.14$$

The resistive force Fr is obtained firstly, by considering the viscous forces fr across the xy planes of a microscopic fluid element dv and secondly, by resolving and summing the components of the resistive forces in the assumed general direction of flow for all the microscopic elements in a much larger macroscopic element, volume Δv, of the porous medium. It follows that:

$$\frac{Fr}{n \Delta v} = - \frac{1}{N} \mu \frac{q}{d^2} \qquad 1.3.15$$

μ is the coefficient of viscosity of the fluid whose flux per unit area through the medium is q. d is the grain diameter, a characteristic of the porous medium, and finally $\frac{1}{N}$ is a constant of proportionality. It follows from equation (1.3.12) that:

$$q = - Nd^2 \frac{\rho}{\mu} \frac{\delta \phi}{\delta L} \qquad 1.3.16$$

Comparing equations (1.3.5) and (1.3.16) it is observed that:

$$K = Nd^2 \rho \frac{1}{\mu} g \qquad 1.3.17$$

It is found that N is a dimensionless parameter which describes the geometrical shape of the internal structure of the medium. The two terms N and d refer exclusively to some property of the medium, Nd^2 is analogous to the, 'intrinsic permeability', of the medium. It is noted that the hydraulic conductivity varies directly with the mass density of the fluid, the gravitational constant g and the intrinsic permeability, and inversely with the viscosity of the fluid. It should, theoretically, be possible to evaluate the intrinsic permeability by investigating the values of N and d. From the considerable literature on this topic it will suffice to emphasise that theoretical determinations are based upon idealized porous media, commonly bundles of parallel tubes. Once the framework of the equation has been erected, factors are built in to allow for the difference in flow characteristics observed in the more usual forms of porous media. Such are the equations of Fair and Hatch (1933) and Kozeny (1927). These equations continue with the use of a single idealized dimension, expressing the size of the porous network, and this also contributes to their limitation for most practical porous media.

1.3 (iii) The applicability of Darcy's Law

Darcy indicated that at high velocities, of the order of 10 cm./sec, of flow of water through the sand column, his relationship no longer applied.

Reynolds number (Re) may be used to indicate kinematically similar systems. It is the ratio between the inertial and resistive forces in the moving fluid. For porous media the index is obtained by an equation of the following type:

$$Re' = \frac{\rho v_n d}{\mu} \qquad 1.3.19$$

v_n is a velocity characteristic of the rate of flow, usually this will be the average flow velocity through the pores, and d is a dimension characteristic of the pore sizes.

Darcy's law applies only to the laminar flow of fluid through porous media, i.e. at low Re'. It does not apply when the flow is turbulent, i.e. at high Re', nor does it apply for a large range of flows between the laminar and the turbulent due to kinetic effects. The Reynolds number applicable to the limiting situation has been investigated by Lindquist (1933 cit Hubbart 1940) who quotes a value of 4.

Scheidegger (1957), however, emphasises the limitations involved in characterising a porous medium by a single dimension and he quotes limiting values of Re' varying from 0.1 - 75.

Let it be assumed, that the critical value of the limit of applicability of Darcy's Law is defined by a Reynolds number of the order of 1. This figure may be used in conjunction with quoted values of hydraulic conductivity in the literature, to compute the order of pore size above which, for that particular result, Darcy's Law no longer holds. Boelter (1965) records the hydraulic conductivity of a peat as 10^{-3} cm./second. This is one of the highest values quoted. The porosity and the hydraulic gradient need to be known to permit the limiting pore size to be determined. Assuming a hydraulic gradient of 10 cm./cm and a porosity n of 0.5, (these values are somewhat unlikely, but they are weighted in favour of a final value of pore size below the actual limit) it follows that:

$$R_e' = \frac{\rho}{\mu} v_n d = \frac{\rho}{\mu} \frac{q}{n} d = \frac{\rho}{\mu} \frac{K \text{ grad } \phi}{n} d \qquad 1.3.18$$

Using the values quoted, and assuming the temperature of the water to be 20°C, it is found that the limiting pore diameter:

$$d = \frac{\mu}{\rho} \frac{n \quad 1}{K \text{ grad } \phi} \cong \frac{0.01 \quad 0.5 \quad 1}{1 \quad 10^{-3} \quad 10} = 0.5 \text{ cm.}$$

Whilst this result is approximate an idea is obtained of the order of limiting sizes. It is unlikely that pores as large as this occur in the peat massif, and there is therefore justification for assuming that the flow conditions that are imposed on peat are well within the range of applicability.

Swartzendruber (1966) and (1968) reviewed the literature dealing with cases of nonproportionality of the soil water movement to the hydraulic gradient. The reasons that have been used to account for the variations are: experimental errors or artifacts, such as the removal of entrapped air; the swelling and shrinking of the soil colloids and the growth of micro-organisms; the interaction between clay and water which can alter the properties of the water considerably, in particular, the soil-water viscosity behaves in a non-Newtonian manner by decreasing with increased rate of shear; the flow of water through porous media can induce an electrical potential difference which counters the flow of water, this is called the streaming potential and finally the shifting of soil particles, or their re-orientation within the medium has also been suggested. Swartzendruber points out that the evidence in support of the validity of Darcy's law is restricted. The often quoted evidence in support of the law by Fancher and Lewis (1933) was obtained with sands and sandstones. He recommends that Darcy's law should be used as a working-hypothesis. The possibility of nonproportionality should, however, be kept in mind as a possible explanation of discrepancies between Darcy-based flow theory and actual experiments.

The selection of methods for the determination of the hydraulic conductivity of peat was based, in the absence of evidence to the contrary, upon the hypothesis that Darcy's law was valid for the flow of water through peat.

SECTION 1.4

The Estimation of Hydraulic Conductivity

1.4 (i) Introduction

A knowledge of hydraulic conductivity is required in most investigations of subsurface seepage, and a reasonably accurate value is therefore desirable.

The conductivity may be estimated by laboratory techniques, requiring the removal of a sample from the site and its transportation into the laboratory, or by field techniques. One is intuitively aware that the removal of a sample of soil from the ground will initiate irreversible changes of the parameters affecting the conductivity, for example, the porosity or the moisture content. Laboratory methods utilise among the smallest samples of any methods and these may not be representative for seepage determination. Finally, it has been suggested that the quality of the water used for the estimation may give rise to variations in the conductivity. This could be relevant to the laboratory estimations which, in general, would use water of a different quality to that found in situ.

The advantage of laboratory methods lies in the control and analysis of the experiments. Rectilinear flow is established in laboratory methods, and this permits the use of simple relations to estimate the conductivity. Field methods, on the other hand, cannot rely upon the establishment of a simple rectilinear pattern, and the relations used to compute the conductivity are more complicated and rely upon the constancy of boundary conditions which cannot always be easily demonstrated. Field techniques do have the advantage over the laboratory methods, that the conductivity of larger, and therefore more representative, samples of soil may be determined, and of course they utilize and endeavour to maintain the natural conditions.

Consider firstly the techniques commonly used in the laboratory and secondly the field methods with a view to illuminating a discussion of the relative merits of the methods available.

1.4 (ii) The estimation of hydraulic conductivity in the laboratory

The most straightforward procedure in the laboratory is to reproduce the experiment of Darcy, using the desired porous medium. In the case of studies of a structureless, isotropic, unconsolidated medium, the sample to be used may be built up in the laboratory itself, for instance a column of sand may be built up. With layered soils, however, it is common practice to obtain the sample, usually cylindrical, from the field. A technology has arisen which investigates the design of instruments to sample the soils in a reasonably undisturbed state.

The sample so obtained may be subjected to the experiment of Darcy (fig. 1.3.1), though the apparatus is sometimes modified slightly. The experiment which coincides with the Darcian situation is better known as, 'the constant head method', (see for example fig.3.4.1.page 174. The formula to determine conductivity is the same form as that derived by Darcy. Thus:

$$K = \frac{Q}{st} \frac{L}{\Delta H} \qquad 1.4.1$$

Alternatively, the head may be varied in a regular manner during the experiment. The method commonly used is termed, 'the falling head method'. This is similar to the initial experiment of Darcy, however, it differs in that water does not enter the top of the sample at constant head but at a diminishing head. This is arranged by connecting the top face of the sample to a standpipe of any desired diameter, though accuracy and rapidity of determination suggest a narrow dimension. Water standing in the pipe to a head H_1 at time T_1 above the outflow is allowed to percolate through the sample, and readings of the relative levels Hi, Hj etc. at time intervals Ti, Tj, are made. Calculations are based on the following formula:

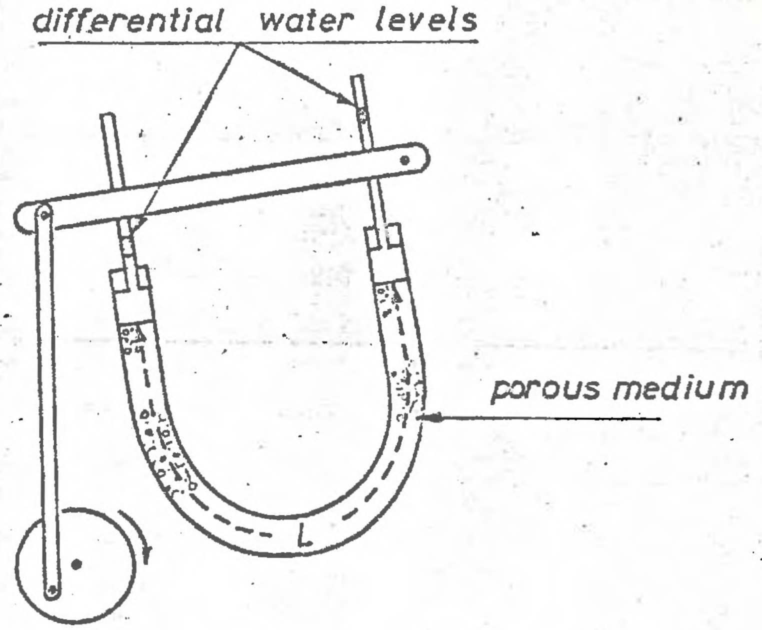
$$K = \frac{aL}{s(Tj - Ti)} \ Ln \frac{Hi}{Hj} \qquad 1.4.2$$

a represents the cross sectional area of the standpipe and s the cross sectional area of the sample normal to the flow.

These two experiments embody the principles of the main laboratory methods; though many modifications to experimental techniques have been described, for example, sampling techniques have been improved, commonly methods seek to use the sampler itself as the container used in the conductivity determination.

Bjerrum and Huder (1957) describe a modification which utilises the sampler, and by means of extra apparatus it is possible to re-create the physical forces that the sample experienced <u>in situ</u>, conductivity is then determined by constant head or falling head techniques. This is similar to the provisions available in the triaxial apparatus of most soil laboratories. The former apparatus was designed to be used on unsaturated clays, where, by means of application of pore water pressure, it was possible to compress and dissolve the entrained air or gas, and thus estimate the conductivity of the saturated clay.

The method described by Childs and Poulovassilis (1960) consists of a u-tube filled with a saturated porous medium of length L, fig. 1.4.1 . The tube is rocked from side to side at a fixed frequency. The water in the medium moves in response to the motion, at the same frequency, but at a different phase. The following equation relates



The Oscillating Permeameter

Fig (1.4.1)

the conductivity and the phase difference $(\pi - \theta)$.

$$\operatorname{Tan} \theta = - \frac{\pi a L}{K b t} \qquad 1.4.3$$

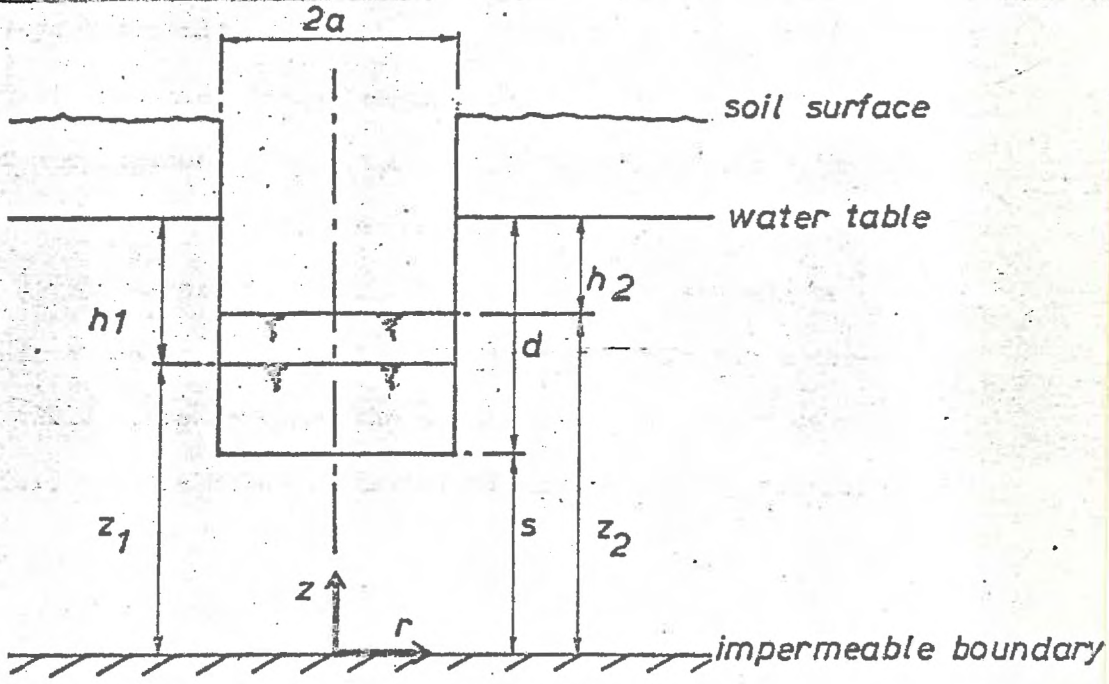
$\frac{a}{b}$ is the ratio between the cross sectional area of the standpipes and the u-tube.

1.4(iii) <u>The estimation of hydraulic conductivity in the field</u>

In exceptional circumstances the topography of the area may allow the conductivity of a soil to be determined from a knowledge of; the bulk flow into a drain or a river, the average hydraulic gradient across the area, in the assumed direction of flow, and the length of the flow path. Alternatively, an area may be chosen and installed with artificial drains in an attempt to impose a similar hydraulic situation to the one described above.

Consider the auger hole method(fig. 1.4.2) where a hole is augered vertically down through the soil horizons. Suppose that it has a radius a, and terminates at a distance d below the water table. The augered hole will begin to fill with water which seeps in through the base, and which may be seen to be running down the side walls due to lateral seepage. After some time, which may be days, the water level in the well or auger hole will have recovered to the level of the water table. During its recovery the water table in the vicinity of the well may be affected by the lateral seepage, but eventually the situation will return to its initial equilibrium.

Two dynamic situations may be imposed. The well may be pumped down to a fixed level, and thereafter allowed to recover naturally, or this level may be maintained by periodic or continuous removal of the inflowing water. In the latter case, the water table



The Auger Hole Method

Fig (1.4.2)

will fall in the vicinity of the well and the displacement will theoretically extend sideways asymptotically to infinity. However, measurement of the water table level at distances where the displacement is not great, and the rate of change of displacement negligible, may be used together with the rate of inflow to the well Q to compute the conductivity.

Dupuit (1863) developed the first relationship between the inclination of a free water surface, and the resulting flow to a continuously pumped well. This involved the assumptions that the flow to the well was radial, and the well penetrated to an impermeable boundary, thus:

$$Q = \frac{\pi K(h_2^2 - h_1^2)}{\ln \frac{r_2}{r_1}} \qquad 1.4.4$$

Q is the rate at which the water is removed, h_1 and h_2 are the depths of the free water surface above the impermeable boundary at radii r_1 and r_2 respectively.

Childs (1969) discussed the further development of equations for continuously pumped wells at equilibrium. Alternatively, the conductivity may be determined during the transient stage when the free water surface around a pumped well is being depressed. Luthin (1957) and Childs (1969) refer to the solution of Theis (1935), namely:

$$\text{Log } h = \log \frac{Q}{4\pi KL} + W(n) \qquad 1.4.5$$

$$\text{Log } n = \log \frac{Y}{4KL} + \log \frac{r^2}{t} \qquad 1.4.6$$

h is the drawdown at time t from the commencement of pumping and at a distance r from the axis of the well which is pumped at a constant rate Q. W(n) is an exponential function, determinable from Jahnke and

Emde's mathematical tables (1938) and Y is the specific yield of the aquifer. Curves are drawn of log. h plotted against $\frac{r^2}{t}$, and log W(n) plotted against log (n). The horizontal and vertical displacements, observed when the two curves are overlain and manouvered to obtain the best fit, are equivalent to log $(\frac{Y}{4KL})$ and log $(\frac{Q}{4\pi KL})$ respectively from which K may be found.

The conductivity may also be determined by observing the recovery of the water level in an auger hole after it has been initially drawn down. Luthin (1957) gives a detailed review of these methods and their equations, the relevent parts of the review are summarised as follows.

Diserens (1934) developed equations relating the pattern of recovery of the water level to the hydraulic potential and the conductivity.

Hooghoudt (1936) improved upon the approach of Diserens. Hooghoudt's equation follows:

$$K = \frac{2.3a \quad S_H}{(2d + a)} \log_{10} \frac{h}{h_2} \qquad 1.4.7$$

The terminology is as detailed in Fig. 1.4.2. S_H was an empirically determined shape factor where:

$$S_H = \frac{ad}{0.19} \qquad 1.4.8$$

Hooghoudt assumed that the direction of flow into the well was normal to the walls. The equation was considered to be applicable only during the period between the hole being initially empty and one quarter full. Ernst (1950) derived the following equation:

$$K = \frac{4000}{(20 + \frac{d}{a})(2 - \frac{h}{d})} \frac{a}{h} \frac{\Delta h}{\Delta t} \qquad 1.4.9$$

h is the head between the water table and the water level in the well, of depth d, at the time t, and $\frac{\Delta h}{\Delta t}$ is the rate of rise of the water level at that time.

Kirkham and van Bavel (1948) presented a mathematical solution for the determination of the conductivity, using an auger hole which terminated at its lower end upon an impermeable boundary. The solution, based upon a development of the Laplace equation, using the known potentials and solving for the assumed boundary conditions, is:

$$K = \frac{\pi^2 a}{16d} \frac{1}{S} \frac{\Delta h}{\Delta t} \qquad 1.4.10$$

Δh is the incremental change of water levels during the recorded time Δt. S is a shape factor dependant wholly upon the geometry of the system. Re-arranging equation (1.4.10) thus:

$$\frac{Sd}{\Delta h} = \frac{\pi^2 a}{16K \Delta t} \qquad 1.4.11$$

The dimensionless nature of S may be deduced by equating the coefficients in this equation. The shape factor may be theoretically determined, for the situation of an auger hole terminating upon the impermeable barrier, by an equation presented in the paper referred to, and approximated by the following equation:

$$S \simeq \cos \frac{\pi(d - h)}{2d} \frac{K_1\left(\frac{\pi a}{2d}\right)}{K_0\left(\frac{\pi a}{2d}\right)} \qquad 1.4.12$$

K_1 and K_0 are Bessel functions the values of which may be obtained from mathematical tables. It was pointed out that the shape factor S may be determined indirectly by using a three dimensional electrical analogue of the flow problem (this approach may also be used to determine a form of shape factor for an auger hole terminating above an impermeable boundary). The use of an electrical analogue does not permit the dimensionless shape factor S to be determined directly. It does, however, allow for the determination of a modified shape factor, 'A', which is related to the shape factor S by the following relationship:

$$A = \frac{16ad}{\pi h} S \qquad 1.4.13$$

It follows from this equation that A has the dimensions of length. Substitution of equation (1.4.13) into equation (1.4.10) gives the following relationship:

$$K = \frac{\pi a^2}{Ah} \frac{\Delta h}{\Delta t} \qquad 1.4.14$$

Van Bavel and Kirkham (1948) presented values of the shape factor A. These were plotted against the ratio between the radius of the auger hole a, and its depth d as a series of curves. The curves represented different positions of the auger hole in relation to the impermeable boundary. In this way, were derived values of A for a hole of radius 5 cms. The values corresponding to holes of different radii were to be obtained by multiplying the factor A by the ratio between the desired radius and the given radius of 5 cm. Thus:

$$A_a = A_{(5)} \frac{a}{5} \qquad 1.4.15$$

Equation (1.4.10) is limited in the range of levels over which it is applicable. As a result, van Bavel and Kirkham (1948) proposed the use of a more general equation applicable to the two situations of an auger hole, with regard to the impermeable boundary, previously mentioned. Thus:

$$K = -2.303 \frac{\pi a^2}{A} \tan \beta \qquad 1.4.16$$

the β is the slope of the log h (on the y-axis) and t (on the x-axis) graph.

Johnson, Frevert and Evans (1952) presented a simplified procedure for determining conductivity. They stipulated the use of an auger hole of 2" radius and reproduced the graph of van Bavel and Kirkham (1948). Since 2" is approximately 5 cms, the values of A, stated in inches this time, were to be directly applicable to the suggested field procedure.

Boersma (1965) reproduced the approximations of Johnson et. al., but nowhere does he specify that the reproduced curve is limited only to a situation where an auger hole of 2" or 5 cms. radius is used. The equation from which the conductivity is computed is equation (1.4.14).

Luthin (1957) reproduced the original curves of van Bavel and Kirkham. The ordinate, instead of being quoted in terms of the shape factor A, is given as a dimensionless parameter which is termed $\frac{S'}{a}$. It is evident that the values for this ordinate are obtained by dividing the original shape factor A by its relevant radius which was 5 cms. The equation which is recommended for use with this shape factor is identical to equation (1.4.10) which follows in its modified form:

$$K = \frac{\pi^2}{16d} \frac{a}{S'} \frac{\Delta h}{\Delta t} \qquad 1.4.17$$

Reconsider, however, that:

$$K = \frac{\pi a^2}{Ah} \frac{\Delta h}{\Delta t} \qquad 1.4.14$$

It is evident that Luthin has confused the shape factors S and A. This can be demonstrated by comparing the equations (1.4.14) and (1.4.17) and by substituting A for S' in equation (1.4.17) whence it follows that:

$$h = \frac{16ad^2}{\pi}$$

where h is a variable, and the term on the right hand side is constant for a particular auger hole. The actual relationship that exists between the two shape factors, S and A, was given in equation (1.4.13).

Hooghoudt (1936) and Ernst (1950) extended their equations to the case of layered soils. Hooghoudt (1936) assumed that the contribution to the rate of rise from each of the layers, may be deduced by a similar reasoning to that which he applied to the isotropic soil. The contribution from a layer, of thickness L_1, and having conductivity

K_1, through which the auger hole passed is:

$$\frac{dy}{dt} = -K_1 \frac{2L_1}{a} \frac{h}{S} \qquad 1.4.15$$

In this equation S the shape factor, is the same as that used in the earlier equation developed by Hooghoudt for isotropic soils. Also, the flow of water, to the auger hole, is considered to be normal to the walls of the well. The sum of the contributions from other layers may be integrated to give the general equation of Hooghoudt:

$$\frac{2K_1L_1 + 2K_2L_2 + \ldots KnLn}{2.3aS} = \frac{\log_{10} h_1/h_2}{t_2 - t_1} \qquad 1.4.16$$

In this solution the auger hole terminates on the boundary between the (n-1)th., and the nth. layers.

Ernst (1950) developed his standard equation to determine the conductivity in two auger holes of different depth. The first, and shallower hole, of depth d_1, terminated well within the first layer. A standard experiment provided a reasonably accurate value of the conductivity K_1 for that layer. The second hole, of depth d_2, penetrated through the first layer, and terminated well within the second layer. The average value of conductivity, determined by a standard experiment was used together with the following equation to compute the conductivity, K_2, for the second layer:

$$K_1 d_1 + K_2(d_2 - d_1) = Kd_2 \qquad 1.4.17$$

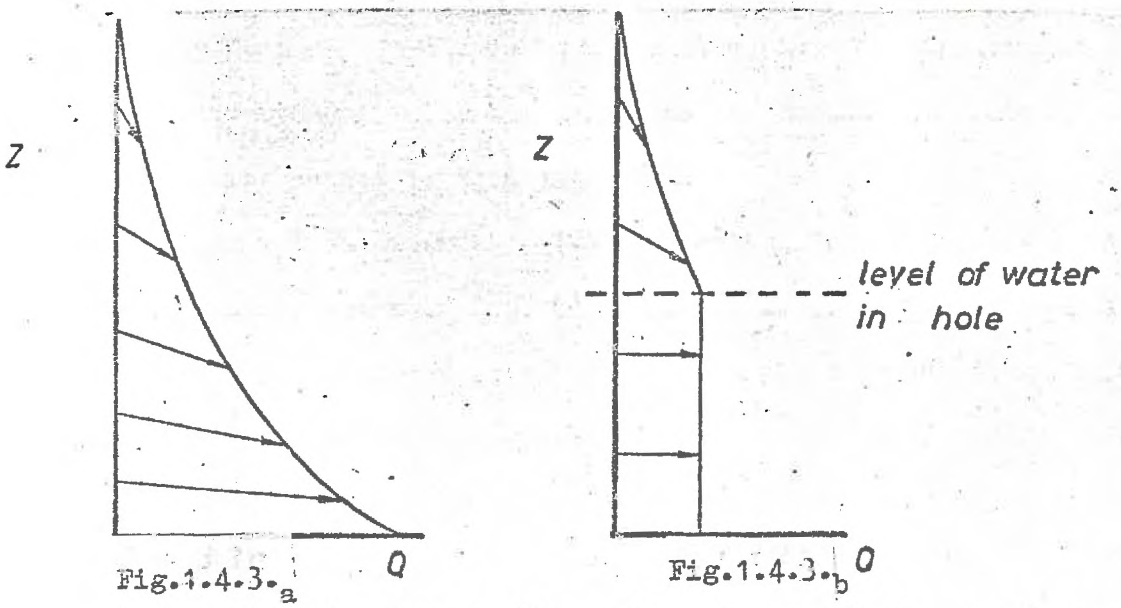
Maasland (1956) extended the theory developed by Kirkham and van Bavel (1948) to the case of a single layered anisotropic soil. The anisotropy is limited to two dimensions, the vertical and horizontal. It is, however, necessary to know the ratio existing between the conductivities in these two directions in order that their true values may be determined.

1.4(iii)a <u>Discussion of the auger hole solutions for estimating conductivity</u>

Kirkham and van Bavel's (1948) solution is attractive since it is an almost exact solution of the Laplace equation. The slight discrepancies arise in the analytical determinations of the shape factor, where, for convenience, terms in the Fourier series are ignored. They calculated the errors arising from this omission, and have shown them to be negligible.

Their analysis confirms the suspicion that the rate of flow through the walls of the auger hole is not uniform with depth at any one time. Also, the rate of flow at any one depth varies with the recovery. This may be demonstrated graphically. The two graphs, figs. (1.4.3 a and b) relate the rate of flow into the hole q, as abscissa to the depth from the base of the hole z, as ordinate. Case (a) applies to an empty auger hole immediately following drawdown. Case (b) applies to an auger hole halfway through the recovery. It is clear from the two curves that not only does the rate of flow vary in time and with depth but its direction varies also. The value of hydraulic conductivity determined for an isotropic soil will be unaffected by this differential variation, but in layered soils the conductivity will be affected by the undeterminable flow-rate of the water seeping into the hole.

Hooghoudt's equation assumes a uniform potential gradient in the vicinity of the auger hole walls, however, Kirkham and van Bavel (1948) demonstrated that the potential gradient varies with depth, being greatest near the base of the auger hole. The same criticism must be made of Ernst's method for layered soils, since he relies upon two values of conductivity, one of which is apparent in the two layered case, and cannot be certainly known unless the variation of potential with depth is also known. All the solutions use shape factors which attempt to describe the geometrical hydraulic situation of the auger hole.



the arrows represent the order of magnitude of the velocity and direction of flow into the auger hole

Sketches showing the relation between the rate and direction of flow of water into an auger hole with depth   case a hole empty
case b .. .. half full

Fig (1.4.3)

Hooghoudt's shape factor ignores the proximity of the impermeable boundary, and the variation in the geometry as the water level rises in the hole. Van Bavel and Kirkham (1948) have shown that the shape factor is on occasions doubled as the hole fills from being empty to half full. It could be that the necessity for the limited ranges of recovery over which the solutions are applicable arises from the non-linearity caused by this phenomenon.

Van Bavel and Kirkham (1948) present values of the shape factor for the hole empty, and half full in an isotropic soil, but the variation between these two situations is not defined, and, as noted above, is too large to allow precise determinations of conductivity to be made. For layered soils, Hooghoudt (1936) continues to use the same shape factor. The shape factors used by Ernst (1950) in his analysis do allow for variation with rising water level, but cannot adequately describe the variation in a layered soil. A similar argument applies to the theory of Kirkham and van Bavel (1948). The analysis of Maasland is, in practice, not suitable since the requirement that the accurate prior determination of the anisotropic ratio be known, presents major problems. It does however, have the virtue of illustrating the bias towards a determination of horizontal, rather than vertical, conductivity by the auger hole method.

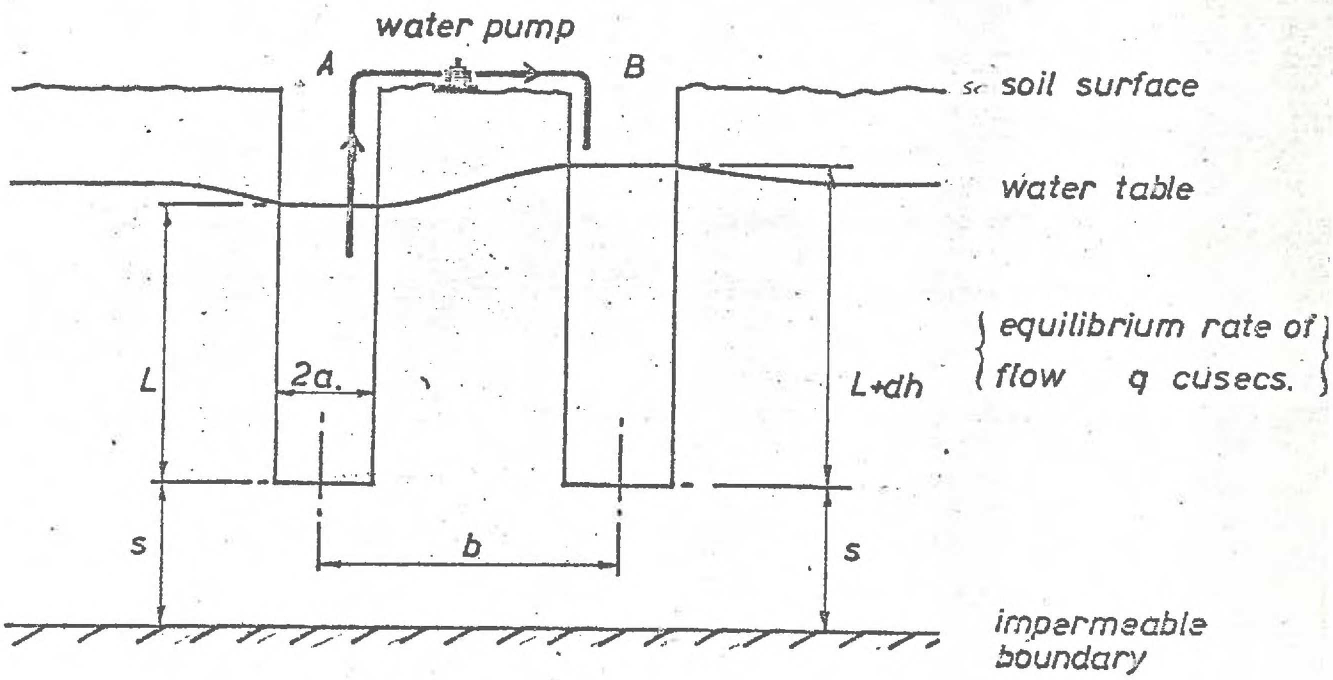
It is pointed out by Kirkham and van Bavel (1948) that, if the auger hole penetrates below the impermeable boundary, and all measurements are confined to the period when this portion is filling up, there will be no variation of the shape factor, all other things remaining sensibly constant. This, however, would not remove any bias that flow may have for particular regions, with the result that the shape factor would still be inaccurate when used for a layered soil. This line of argument raises the rather philosophical concept of the degree of impermeability necessary for a boundary to be deemed impermeable. Clearly there is a relation between the conductivities of the soil concerned and that of any lower soil layer which could be used as an index to the acceptance of the term impermeable. There is no positive answer to this. The literature suggests that provided the discontinuity is at about twice the depth of the auger hole, and provided the conductivity below the discontinuity is at least less than the layer under consideration, the value of the shape factor will be fairly accurate.

The Two Well Method of Childs (1952)

Consider fig. (1.4.4). A and B represent two similar wells augered into an isotropic soil. At some distance S below the base of the wells there is an impermeable boundary. At some time following their installation the water in the wells will have reached an equilibrium with the surrounding water table. A hydraulic potential gradient is established between the wells, by pumping water at a known rate from one to the other. In the event that the potential difference is small the peturbation of the water table in the vicinity of the wells will also be small, with the result that the plane of seepage from one well to the other will be approximately horizontal. Childs has demonstrated that the flow pattern established is similar to the classical pattern arising between a source and sink, distance b apart. Using a solution presented by Smythe (1950), Childs developed the following equation:

$$K = \left(\frac{q}{\pi l \Delta h}\right) \cosh^{-1} \frac{b}{2a} \qquad 1.4.18$$

Since this equation applies only to flow in a horizontal plane it is necessary that the wells should penetrate to the impermeable layer. When the wells terminate above the layer, a certain relaxation may be



The Two Well Method of Childs

Fig (1.4.4)

visualised as occurring in the horizontal flow pattern. The disturbance from the horizontal will be most marked at the base of the wells. Childs argued that this would allow for a greater flow than anticipated which, he suggested, could be accommodated by adding for this terms to equation (1.4.18).

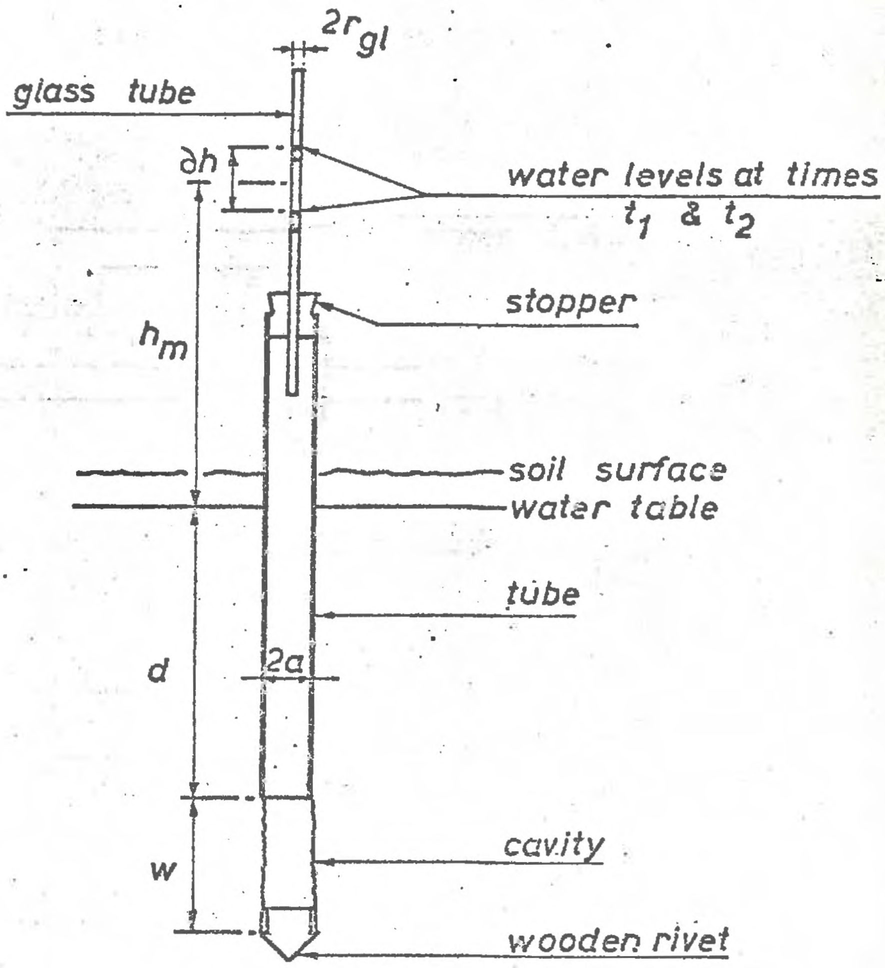
Childs (1952) has shown that the two dimensional horizontal anisotropy of a soil may be investigated by three wells situated at the nodes of an imaginary 45° triangle, and the three dimensionless anisotropy by a combination of the two well method and the seepage tube method, yet to be described.

Smiles and Youngs (1963) pointed out that the two well method was just one example of a radially symmetrical array of wells. They developed equations for multiple wells, situated around the circumference of a circle.

The Seepage Tube Method

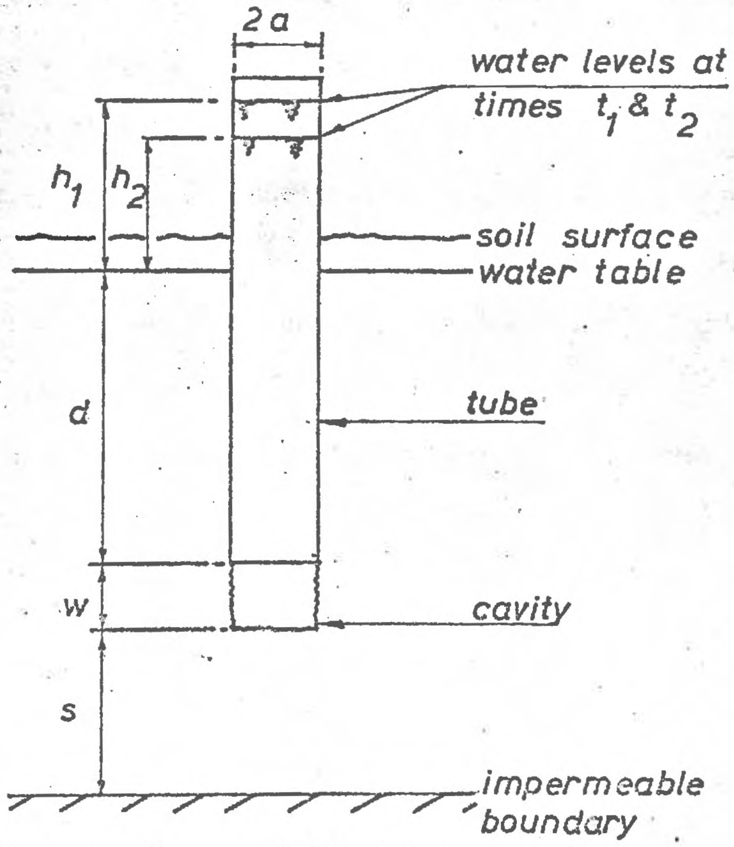
The final method for consideration is the seepage tube method, see figs. (1.4.5) and (1.4.6). A well is augered into the soil to some depth below the water table, and is tightly lined by an impervious tube. The augered hole may continue below the end of this tube, creating a cavity in the soil into which water may seep.

Khafagi (1944) devised two procedures for inserting tubes and for forming cavities below them. For the two procedures he presented two experimental equations. The tube shown in fig. (1.4.5) was driven to the required depth, after which a wooden rivet, that fitted into the lower end, was hammered out to form the cavity of length d, and radius r_c, in the soil. In this case the rivet was expendable; for the



Seepage Tube Method (Khafagi)

Fig (1.4.5)



Seepage Tube Method (Kirkham)

Fig (1.4.6)

situation where the cavity was formed by a rivet which was then removed, Khafagi presented a different formula. After installation, the water level in the tube will come to equilibrium with the water in the soil surrounding the cavity. Khafagi made the assumption that, in the initial state, this equilibrium level would be identical to the level of the water table. For the purposes of the experiment, the seepage tube was filled with water. A tight fitting rubber bung, through which a thin graduated glass tube passed, was then fitted tightly into the neck of the seepage tube. Positioning of the bung displaced water into the glass tube. A record was kept of the subsequent falling heights of the water h_1, h_2 in the glass tube, relative to the water table, at the respective times t_1, t_2 etc. The equation applicable when the rivet was left in place is:

$$K = \frac{(r_{gL})^2}{2hm\left(\frac{d/4 + w + a}{4}\right)} \frac{dh}{dt} \qquad 1.4.19$$

where dh is the head interval $(h_1 - h_2)$, hm is the mean head $(h_1+h_2)/2$ and dt is the time interval (t_2-t_1).

In the event that the rivet is removed, the equation proposed is:

$$K = \frac{(r_{gL})^2}{8hm} \frac{dh}{dt} \qquad 1.4.20$$

It will be noted that no account was taken of the presence of an impermeable boundary.

Kirkham (1945) presented an equation which related conductivity, hydraulic potential and resultant seepage rate into a seepage tube where the cavity was not augered beyond the end of the tube. The equation was based upon an exact analysis of the Laplace equation for the hydraulic potential stated, and the boundary conditions specified. The analysis is not dissimilar to the derivation of the exact equation for the auger hole method. Two situations were defined. In the first, the water level in the tube after being raised by adding water, was allowed to re-establish its own equilibrium with the water table, or more particularly with that potential naturally existing in the vicinity of the cavity of zero length, see fig. (1.4.6).

Kirkham derived the following equation:

$$K = \frac{\pi a^2 \ln h_1/h_2}{A(a,d,s)(t_2 - t_1)} \qquad 1.4.21$$

A(a,d,s) is a shape factor determined by electrical analogy and having the dimensions of length. It depends upon the geometry of the tube and its relation to the impermeable layer and the water table.

The alternative procedure required that the head h be maintained constant and the rate of supply to the pipe q, which at equilibrium was also constant, be measured. The following formula is applicable to that case:

$$K = \frac{q}{A(a,d,s)h} \qquad 1.4.22$$

Frevert and Kirkham (1943) pointed out that there was no analytical difference between the case where added water flows from the tube to the soil and that in which flow takes place from the soil to the tube, to replace water removed.

Luthin and Kirkham (1949) described what they termed, 'a piezometer method'. This used narrow diameter tubes, but the important point was demonstrated that the equations (1.4.21) and (1.4.22) could be easily modified, to allow for the case where a cavity, of known length w, extended below the tube. A(a,d,s) was altered to become A(a,d,s,w). Substitution of this term in equations (1.4.21) and (1.4.22) in place of A(a,d,s) gave an exact solution, (see Section 1.7). A limited number of values of $A(a,d,s,w)$ were also given.

Childs (1952) suggested the way this method, with a cavity $w = 0$, could be used in conjunction with the two well method to determine the three dimensional anisotropy of the soil. He also suggested that, provided one assumed anisotropy to occur only between the vertical and horizontal directions i.e. the soil in the horizontal plane is isotropic, its value could be determined by using two different cavity sizes, the second following the first at the same position. Childs showed that:

$$\pi a^2 \beta \mathrm{Ln}\, \frac{h_1}{h_2} = K_H \; A(a,d,s,w,\beta)\; dt \qquad 1.4.23$$

where β is the linear ratio existing between the vertical and horizontal conductivities K_V and K_H, and dt is the time interval $t_2 - t_1$. Using the two cavities, of radii a and a', the following relationship applies:

$$a'^2 dt_{(a)} \; A(a,d,s,w,\beta) \mathrm{Ln}\, \frac{h_{1(a')}}{h_{2(a')}} = a^2 dt_{(a')} \; A(a',d,s,w,\beta) \mathrm{Ln}\, \frac{h_{1(a)}}{h_{2(a)}} \qquad 1.4.24$$

from which it is possible to find a value for β. Substitution of this value in equation (1.4.23) permits the estimation of K_H. Now since:

$$\beta = \left(\frac{K_H}{K_V}\right)^{1/2} \qquad 1.4.25$$

the true value of K_V may also be estimated.

Collis George and R.N. Evans (1964) described a two well method for determining the hydraulic conductivity of discrete layers, in layered saturated soils, in the field. The principle was essentially the same as that used in the two well experiment of Childs (1952) and Childs, Cole and Edwards (1953) in that a sink and source were created, by the removal of a quantity of water from one well and the addition of an equivalent quantity to the other. Two modifications were introduced. First, the flat base of the well was sealed off to prevent vertical flow into the well, and secondly, the wells were lined from the surface for part of their depth with tightly fitting tubes. The resulting surface of seepage below a tube resembled the surface of a cylindrical cavity of length w and diameter 2a.

The two wells were positioned, such that the cavities were at the same depth, in a layered soil, within the same layer and at a distance b apart. The determination followed the re-establishment of equilibrium conditions. Measurements were made of rate of flow $\frac{dQ}{dt}$ and the resultant potential difference dh between the wells when equilibrium conditions had been re-established. The formula used to estimate the horizontal conductivity follows:

$$K_H = \frac{\Omega \frac{dQ}{dt}}{(w + w_e)\ dh} \qquad 1.4.26$$

Ω is a dimensionless calculable factor and w_e is the, 'extra length', which is used to allow for the fact that the flow between the wells is not completely horizontal.

In soils exhibiting anisotropy between the vertical and the horizontal conductivities a determination of the apparent conductivity K' is first made using a seepage tube with a cavity of zero length. The cavity is situated in the middle of the layer under consideration. It follows that:

$$K' = \frac{\pi a^2 \ln \frac{h_1}{h_2}}{A(a,d,s,w)\ (t_2 - t_1)} \qquad 1.4.27$$

which is a re-statement of Kirkham's formula.

The modified form of equation (1.4.26) follows:

$$K_H = \frac{\Omega \frac{dQ}{dt}}{w dh} - \frac{w_{e(isotropic)} K'}{w} \qquad 1.4.28$$

The correction to the cavity length, $w_{e(isotropic)}$ is determined either by an electrical analogue, or by use of nomograms presented in the paper under consideration. It is related to the true correction for anisotropic conditions by the relations:

$$w_{e(anisotropic)} = \frac{1}{\beta} w_{e(isotropic)} \qquad 1.4.29$$

where:

$$\beta = \sqrt{\frac{K_H}{K_V}} = \frac{K_H}{K'} \qquad 1.4.30$$

It is seen from the nomograms that the ratio $\frac{w_e}{w}$ approaches a limiting value, as the ratio between the cavity length and the layer thickness in which the cavity is situated, diminishes.

Care is required when the cavity length approaches the thickness of the layer under consideration, and account must be taken not only of its geometrical relation to the local boundaries, but also of the absolute values of the hydraulic conductivities in the confining layers.

Smiles and Youngs (1965) described the use of several of the field methods, previously mentioned, when they were carried out in a laboratory tank filled with sand. They discussed the resulting values of hydraulic conductivity, determined at the most at nine positions, and using four distinct techniques: the seepage tube with various cavities; the auger hole; the well methods, using two, four and eight wells; and one pumped well with the solution of Theis (1935).

The results suggested that all methods were reasonably consistent, and the choice of individual methods to use in the field would depend only on site conditions and the availability of labour.

SECTION 1.5

The Hydraulic Conductivity of Peat

1.5 (i) Introduction

Peat is a highly compressible organic soil formed from various types of plant remains. There are different types of peat characterised by the predominant plant species from which they are formed. Peat tends to form layers and where the peat has formed on a surface of shallow gradient the layers are approximately horizontal.

The choice of methods which may be suitable for estimating hydraulic conductivity are considered in the following three tables. The first table, section 1.5(ii) is devoted to a comparison of some of the methods used to estimate the hydraulic conductivity of soil. The choice of methods is based upon the need to select methods for peat. The body of the table is divided into two sections. The first forsees the possible disadvantages of the proposed method and the second the possible advantages. The second table, section 1.5(iii) is devoted to a review of the methods which have been used in peat, with comments where relevant. Each reference cited has dealt with more than one method and this, therefore, permits comparison. In the third table, section 1.5(iv) a review is presented, enabling comparison of the values of the hydraulic conductivity of various qualitatively assessed peat types quoted in the literature.

1.5 (ii) COMPARISON OF METHOD CONSIDERED SUITABLE FOR T

Suggested Method	Disadvantage of the Proposed Method
Constant head permeameter	Disturbance of the peat when sampled and t size of sample considered.
Bjerrum Huder permeameter (1957)	" " " "
Large scale field estimations based upon known potential and discharge	Requires situations of particular specific Difficulty of defining the boundaries of i The results lack sensitivity to changes wi the peat.
Pumped Well: Theis (1935)	Consolidation, drainage and aeration of th may alter its physical properties.
Auger Hole: Kirkham and van Bavel (1948)	The result may be dominated by the layer greatest conductivity. The value obtained is only an average for and not anisotropic soils. The size of peat sampled varies with the of the well. The shape factor may varyby as much as 20 half the recovery period for isotropic so is undefinable for layered and anisotropi Installation of a well may itself create systems between layers having different i potential heads.
Two Well Method: Childs (1952)	As above the wells may be independent flo invalidating the result at low differenti The result may be dominated by layers of conductivity. Two different flow systems are establishe peat and out of the peat which may be dif character.
Seepage Tube: Kirkham (1945)	The water table may represent a false up when a tube is installed in a situation w seepage is occurring. The size of peat sampled will in general
Two Tube Method: Collis George (1964) and Evans	The size limitation of the tubes may resu situation in two wholly different peat ty Requires a knowledge of the conductivity the vertical direction in the locality of under consideration.

LE FOR THE ESTIMATION OF THE HYDRAULIC CONDUCTIVITY OF PEAT

d	Advantage of the Proposed Method
ed and the small	The flow within the sample will in general be rectilinear. The simplicity of the imposed boundary conditions. Effects due to variations of absolute pressure or hydraulic gradient may be investigated separately.
"	" " " " " "
specification. ies of flow. anges within	Yields a result which may be applied to the practice and design of systems, ie drainage, which may involve large areas.
on of the peat	After the initial drawdown the flow geometry remains reasonably stable. The large size volume of peat considered enables the effects of irregularities of the peat structure to be ignored.
layer with the	
ge for isotropic	
n the recovery	
as 200% during pic soil and it otropic soils. reate flow rent initial	
nt flow systems erential heads. rs of high	The method could give an average value of conductivity for uniform layered soils. At equilibrium shape factors will also be constant.
blished, into the be different in	
se upper boundary tion where natural	The shape factor remains constant during the experiment and it is not greatly altered by the presence of local discontinuities.
neral be small.	The small size increases the sensitivity to changes within the peat itself, which may be followed by reference to the recovery relationship $\dfrac{\ln \dfrac{h_1}{h_2}}{t_2 - t_1}$ alone. It may be possible to isolate the individual peat types and determine the orders of conductivity for each.
result in their eat types. ivity profile in ty of the soil	An approximately knownflow geometry is imposed. The conductivity may be estimated at any desired value of hydraulic potential.

Rycroft. Part. 2)

HAVE BEEN USED TO ESTIMATE THE HYDRAULIC CONDUCTIVITY OF PEAT

Comments and Recommendations for the particular method

The Auger Hole method gave reproducible results, these compared favourably with values computed from known potential gradients between, in the one case ditches and in the other drains, and the measured outflow. The results from the seepage tube were incon istent, and in general the values quoted were less than those determined by the use of an Auger Hole; it was suggested that this was a direct result of the puddling of peat forming the surface of the cavity. The Auger Hole method was adopted as the standard method to be used.

No significant difference between the values of conductivity measured in the vertical and the horizontal directions.

The results from the pumped well were not constant, due possibly to air entrappment, and were difficult to interpret. The laboratory method also gave results which were not constant.

at Values of conductivity quoted in terms of 'relative conductivity'. This makes the results difficult to compare with other workers. Good agreement between Auger Hole and funnel permeameter.

Conductivity measured by Auger Hole is excessive, so this method is not suitable. Only the 3 other methods used can be relied upon to give accurate results.

1.5 (iii) REVILW OF THE METHODS WHICH HAV

Authors and Dates	Methods Employed	Peat Types
Baden, W. and Eggelsmann, R. (1963)	Auger Hole: Hooghoudt (1936), Ernst (1950). Seepage Tube: Khafagi (1944). Calculations using drain and ditch outflow, and measured potential differences.	Various
Boelter, D. H. (1964)	Seepage Tube: Kirkham (1945) using: a Cavity 10 cm. deep, 3 cm. diameter. b Cavity zero depth, 20 cm. diameter.	Various
Galvin, L. F., Hanrahan, E. T. (1967)	Continuously pumped well, Bjerrum Huber permeameter.	Various
Huikari, O. (1959)	Auger Hole, Funnel permeameter: Malmstrom (1938)	Cultivated Pea
Gebhardt, E. (1967)	Auger Hole: Hooghoudt, Ernst. Seepage Tube: Khafagi. Pumped well and Laboratory methods.	Reed Peats

CTIVITY FOR VARIOUS TYPES OF PEAT QUOTED IN THE LITERATURE

alues of Hydraulic Conductivity cms/sec	Comments and Conclusions from the Results
.89 x 10^{-5} – 2.76 x 10^{-1}, verage 2 x 10^{-3}	Conductivity decreases with increasing volume of solid matter of which the peat is composed. Conductivity decreases with increasing humification, as expressed by von Post and Granlund (1926).
ndrained 1.7 x 10^{-3}, rained 2.6 x 10^{-4}	
x 10^{-3}	
ncultivated 10^{-3}, drained 7.8 x 10^{-4}, ultivated 3.6 x 10^{-5} – 4.8 x 10^{-5}	
ndecomposed *Sphagnum* 73.8 x 10^{-2}, ecomposed Herbaceous 7.5 x 10^{-6}	No significant variation between conductivity in horizontal and vertical directions. Decomposition related to fibre content. (Fibres defined as > 0.1 m.m.). Conductivity increased linearly on a logarithmic scale with the decrease of the percentage of the fibre content (normal scale).
x 10^{-6} at -4 p.s.i. pressure to .2 x 10^{-5} at + 12 p.s.i.	Conductivity varied with the head at which the value was determined.
	Conductivity decreased with increasing solid volume percentage, and increases with increasing fraction of particles > 50 μm. Experimental recoveries in holes 1m. deep by 100 cm^2 area sometimes took two days to recover.
ater tracks, amorphous 2.4 x 10^{-7} ire expanse, fibrous, 9.6 x 10^{-8}	
	Vertical conductivity > horizontal for low humifications (von Post and Granlund). The distinction is non existant for humification in excess of H5.
per 'active layers', 10 - 10^{2}, wer layers 10^{-6} - 10^{-5}	
	Relative conductivity decreases by the order of 100 times as humification (von Post and Granlund) increases from H_{1-2} to H_8.
cms. below surface 5.5 x 10^{-8} - 5.4x10^{-7} cms. below surface 1.2 x 10^{-7} - 2.3x10^{-7}	Hydraulic conductivity decreases with depth below the surface.

REVIEW OF THE VALUES OF HYDRAULIC CON

Author and Date	Methods Employed	Peat Types
Baden, W. and Eggelsmann, R. (1961)	Auger Hole	Various
Baden, W. and Eggelsmann, R. (1963)	Various	Various
Baden, W. and Eggelsmann, R. (1964)	Auger Hole	Raised Bog Peat Undrained-Drained
Eggelsmann, R. (1962)	Auger Hole	Sedge Peat
Eggelsmann, R. and Mäkelä, T. (1964)	Auger Hole	Uncultivated, Drained and Cultivated
Boelter, D.H. (1964)	Seepage Tube: Kirkham	Various peat types
Boelter, D.H. (1969)	Seepage Tube: Kirkham	Various types and degrees of decomposition
Galvin. L.F. and Hanrahan, E.T. (1967)	Bjerrum Huder Permeameter	Various
Gebhardt, E. (1967)	Seepage Tube: Khafagi	Various
Huikari, O. (1959)	Auger Hole Funnel Permeameter	Various
Ingram, H.A.P.	Seepage Tube: Kirkham	Blanket Bog
Mälmstrom, C. (1939)	Constant head permeameter	Various
Romanov, V.V. (1961), tr. (1968)	Auger Hole Flume	Various
Sarasto, J. (1961)	Funnel permeameter: Malmstrom	Sphagnum and Sedge
Sturges, D.L. (1968)	Seepage Tube: Kirkham	Fissured decomposed peat

1.5 (v) Conclusion

From section 1.5(ii) it is concluded that the seepage tube method, using the solution of Kirkham, will be the most successful for the determination of hydraulic conductivity of peat. The review of the various methods used in the field, 1.5(iii) is not conclusive and little inference may be drawn as to which method is the most suitable. Suffice to say that there are no real adverse comments concerning this method. In section 1.5(iv) several points are emphasised. The large overall range of the quoted results, eight orders of magnitude, tends to stand out. It is clear that hydraulic conductivity decreases with increasing humification (the humification may be expressed qualitatively, von Post and Granlund (1926), or as a function of the fibrous remains in the peat, itself a function of the volumetric percentage of solid material present). There are clearly differences of orders of magnitude of conductivity between the upper undecomposed layer, and the lower humified peats, as a result their estimation may require different techniques.

It is felt that the hydraulic conductivity of peat is a physical property of interest in itself. Previous work has suggested that there is no unanimity as to the best way of estimating this property.

It is intended to investigate hydraulic conductivity in peat using the seepage tube method, and the solution of Kirkham. In the following chapter the derivation of Kirkham's formulae will be considered.

SECTION 1.6

The Derivation of Kirkham's Formulae

1.6 (i) Introduction

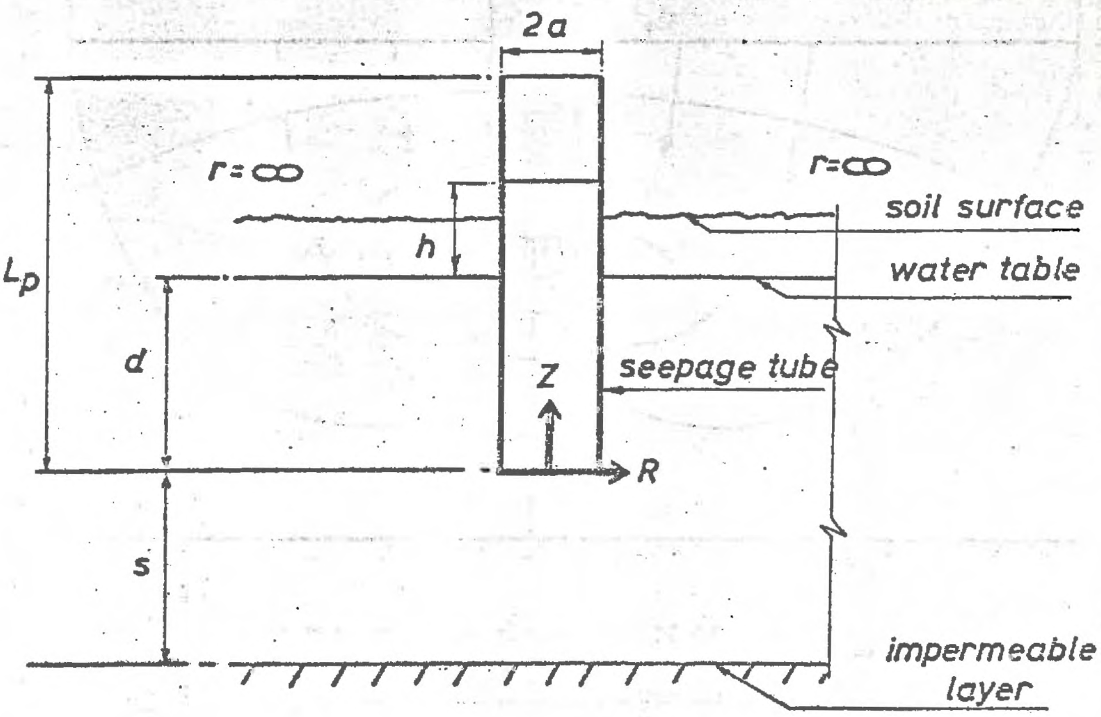
An open ended tube, of length L and diameter 2a is installed in the soil profile to a depth d below the water table fig.(1.6.1). The soil which occupied the space taken by the tube is augered out, usually as the installation proceeds, to leave a circular surface of seepage, the cavity, at the base of the tube. At some time after the installation the water level in the tube will have recovered, it is assumed to the same level as the surrounding water table.

The experiment is performed either by depressing or raising the water level in the tube. The configuration of streamlines and equipotentials is sketched in fig.(1.6.2) for the latter case. This case will be referred to as 'a recharge experiment', to distinguish it from 'the depletion experiment', when water flows from the soil into the tube.

Two equations follow. The first applies to the situation where the water level within the tube is maintained in the displaced position, by the removal or addition of water at the constant rate q, namely:

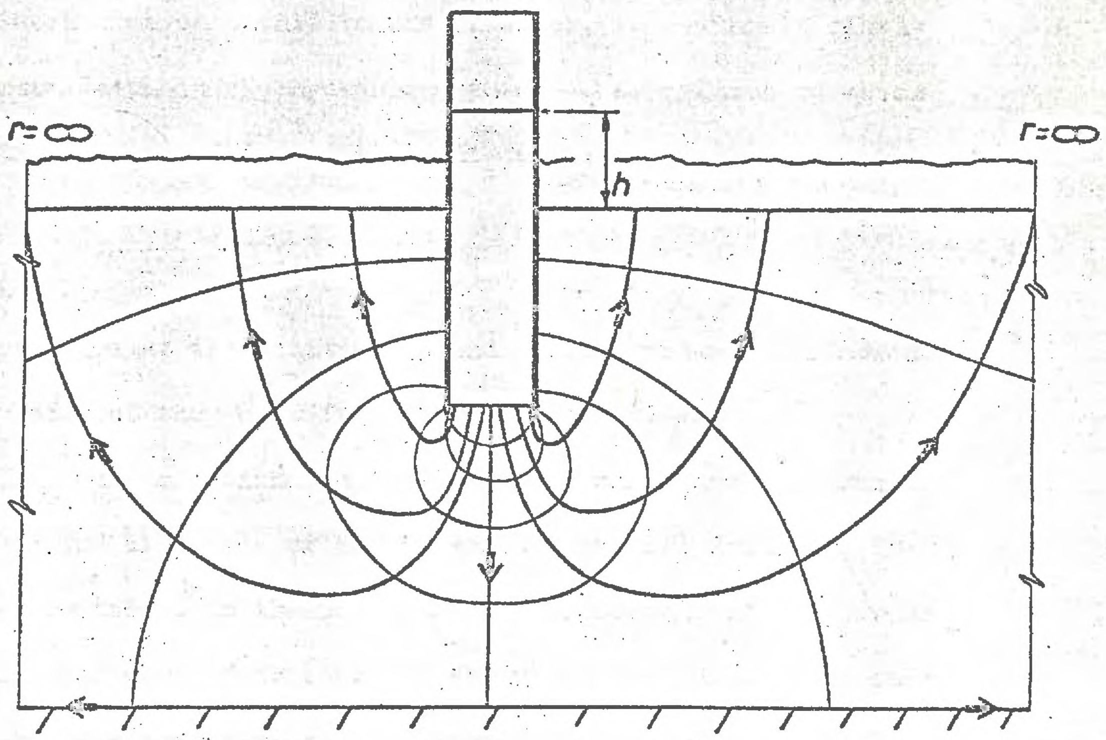
$$K = \frac{q}{A(a.d.s)h} \qquad 1.6.1$$

A(a.d.s) is a geometrical shape factor having the units of length. The second equation applies to the situation where the displaced water level is allowed to return naturally to its former equilibrium position, and the differences of the potential or relative heads h_1, h_2 etc. are recorded at successive time intervals t_1, t_2 etc., namely:



A Seepage tube in situ

Fig (1.6.1)



Sketch of the Configuration of the Streamlines & Equipotentials

Fig (1.6.2)

$$K = \frac{\pi a^2 \ln \frac{h_1}{h_2}}{A(a.d.s)t_2 - t_1}$$

1.6 (ii) The derivation of the equation for the case where the potential head is maintained constant

A hydraulic potential Φ is introduced and defined as the amount of work done on a unit volume of water, in moving it slowly from a chosen datum in the hydraulic system to a position to which the potential refers (Childs 1969). It can be shown that

$$\Phi_{vol} = P + \rho g z \qquad 1.6.3$$

P is the pressure at the position, ρ and g as previously defined and z the elevation of the point from the datum position.

In a porous medium the facility with which the unit volume is moved, in the direction of the steepest potential gradient, is directly proportional to the intrinsic permeability k of the porous medium and inversely proportional to the viscosity of, in this case, water. A similar potential to that expressed by equation (1.6.3) may be written for porous media, namely:

$$\Phi_{vol_P} = \frac{k}{\mu} (P + \rho g z) \qquad 1.6.4$$

A continuity equation may be written for flow in homogenous, isotropic, incompressible porous media, namely:

$$\frac{\delta V_z}{\delta z} + \frac{1}{r} \frac{\delta V_r}{\delta r} = 0 \qquad 1.6.5$$

The above equation is expressed in cylindrical co-ordinates. V_z and V_r are the component velocities of the flow in the respective directions.

Now:

$$V_z = \frac{\delta \Phi}{\delta z} vol_P \quad \text{and} \quad V_r = \frac{\delta \Phi}{\delta r} vol_P \qquad 1.6.6$$

Substitution of the relations in equation (1.6.6) into equation (1.6.5) results in the following expression:

$$\frac{\delta^2 \bar{\Phi}_{vol_P}}{\delta z^2} + \frac{1}{r} \frac{\delta^2 \bar{\Phi}_{vol_P}}{\delta r^2} = 0 \qquad 1.6.7$$

This is a form of the Laplace equation which may be expressed simply as:

$$\Delta^2 \bar{\Phi}_{vol_P} = 0 \qquad 1.6.8$$

Assuming that $\frac{k}{\mu}$ is constant the problem may be simplified to one of solving:

$$\Delta^2 \bar{\Phi}_{vol} = 0 \qquad 1.6.9$$

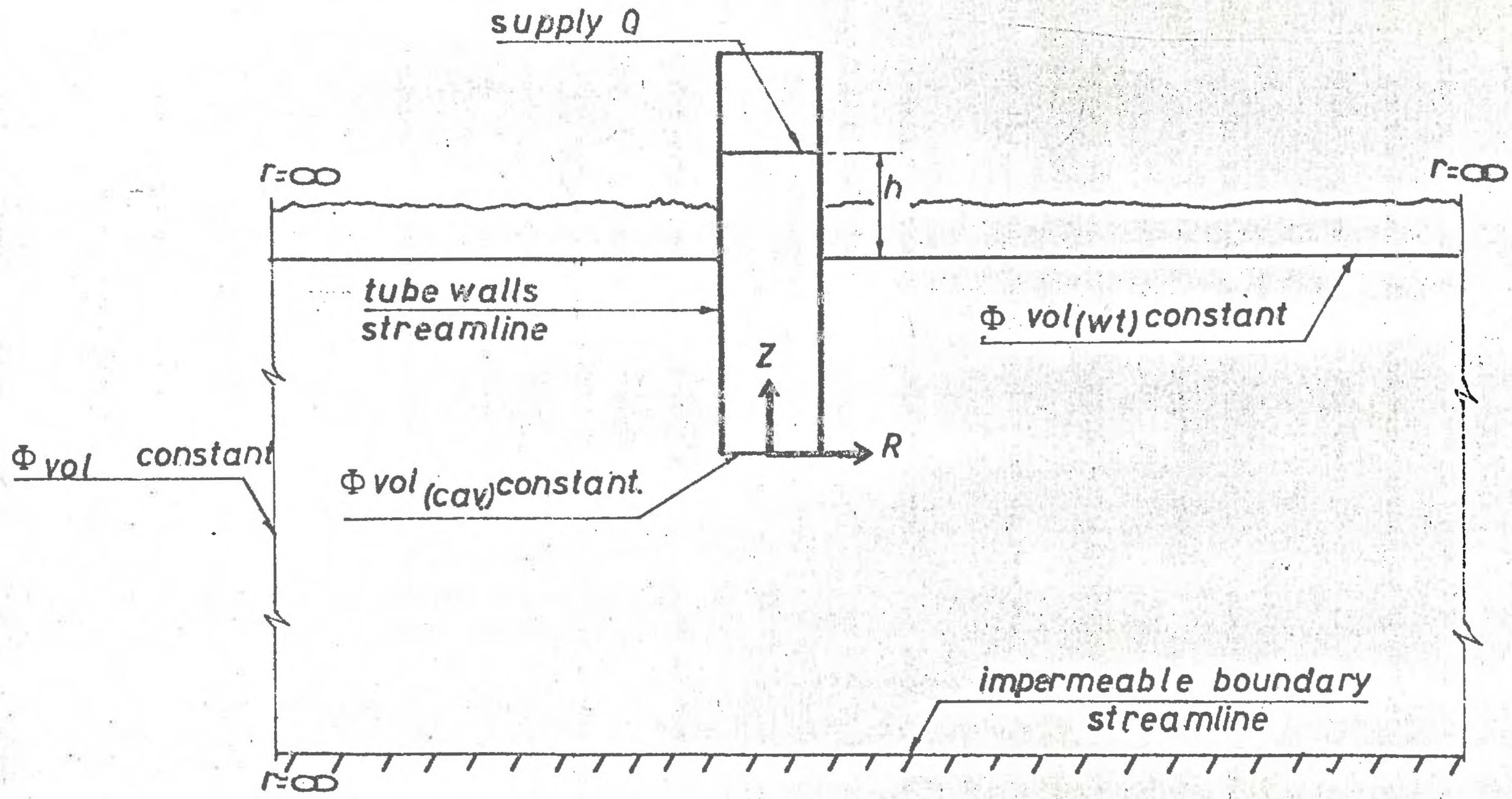
This is a second order differential equation, the solution to which will take the following form.

$$\bar{\Phi}_{vol} = B\emptyset + C \qquad 1.6.10$$

B and C are constants, the dimensions of which may be chosen to allow for the physical characteristics of the fluid and its environment, thus leaving $\emptyset$ as a geometrical function.

The solution to this equation follows from considering the boundary conditions, fig. (1.6.3) which are:

1. The water table is assumed to be initially horizontal and therefore an equipotential. That prior to the disturbance, the water regime is static, and that the water table is unaffected by the dynamic system established. It therefore remains an equipotential surface at all time, $\bar{\Phi}_{vol_P}$ and thus $\bar{\Phi}_{vol}$ remaining constant.



Sketch of the Potential conditions at Equilibrium

Fig (1.6.3)

2. The water surface within the tube and the surface of the cavity at the base of the tube are assumed to be equipotentials, both existing at the same potential.

3. The walls of the tube and the impermeable boundary are stream surfaces. There is no component of flow normal to these surfaces.

4. The flow diminishes to insignificance as $r \rightarrow \infty$.

The potential on the water table referred to the base of the tube as datum, fig. (1.6.1) is:

$$\Phi_{vol_{(wt)}} = Pa + \rho gd \qquad 1.6.11$$

The subscript wt for water table refers to the position at which the potential applies. Pa is the atmospheric pressure.
The potential at the cavity face is:

$$\Phi_{vol_{(cav)}} = Pa + \rho g(d + h) \qquad 1.6.12$$

The following equations follow from considering equations (1.6.10), (1.6.11) and (1.6.12), namely:

$$Pa + \rho gd = B\phi_{(wt)} + C \qquad 1.6.13$$

$$Pa + \rho g(d + h) = B\phi_{(cav)} + C \qquad 1.6.14$$

Solving simultaneously it is found that:

$$B = \frac{\rho gh}{\phi_{(cav)} - \phi_{(wt)}} \qquad 1.6.15$$

and:

$$C = Pa + \rho g d - \frac{\rho g h}{\phi_{(cav)} - \phi_{(wt)}} \phi_{(cav)} \qquad 1.6.16$$

Re-substitution of these equations in equation (1.6.10) gives the following general result:

$$\Phi_{vol_P} = \frac{\rho g(h)}{\phi_{(cav)} - \phi_{(wt)}} \phi + Pa + \rho g d - \frac{\rho g(h)}{\phi_{(cav)} - \phi_{(wt)}} \phi_{(cav)} \qquad 1.6.17$$

The volume rate of flow of water seeping into the pipe $\frac{dq}{dt}$ may be determined by the following expression:

$$\frac{dq}{dt} = \int_a^{\infty} \frac{\delta \Phi}{\delta z}_{vol_P} 2\pi r\, dr \qquad 1.6.18$$

Now:

$$\Phi_{vol_P} = \frac{k}{\mu} \Phi_{vol} \qquad 1.6.19$$

Equation (1.6.19) may be substituted in equation (1.6.17) which relation may then be substituted in equation (1.6.18) to give the following:

$$\frac{dq}{dt} = -\frac{k\rho g}{\mu} \frac{h}{\phi_{(cav)} - \phi_{(wt)}} \int_a^{\infty} \frac{\delta \phi}{\delta z} 2\pi r\, dr \qquad 1.6.20$$

The geometry of the system and the configuration of streamlines and equipotentials are independant of the imposed potential. The geometry is constant for any one experiment. Equation (1.6.20) may be simplified as follows:

$$\frac{dq}{dt} = \frac{k\rho g}{\mu} h\, A(a.d.s) \qquad 1.6.21$$

Where

$$A(a.d.s) = \frac{-1}{\phi_{(cav)} - \phi_{(wt)}} \int_a^{\infty} \frac{\delta \phi}{\delta z} \begin{matrix} 2\pi r\, dr \\ z = d \end{matrix} \qquad 1.6.22$$

Integration of equation (1.6.21) between the time limits t_1 and t_2 leads to the following expression for q the measured flow during the time interval $t_2 - t_1$.

$$Q = \frac{k\rho g}{\mu} h\, A(a.d.s)(t_2 - t_1) \qquad 1.6.23$$

the rate of flow

$$q = \frac{Q}{t_2 - t_1} \qquad 1.6.24$$

furthermore:

$$K = \frac{k\rho g}{\mu} \qquad 1.6.25$$

Thus:

$$K = \frac{q}{A(a.d.s)h} \qquad 1.6.1$$

1.6(iii) The derivation of the equation for the case of a natural recovery following depletion or recharge

The potential difference h established initially between the water table level and the level within the tube, diminishes with the recovery.

Suppose that the water level within the tube stands at some distance h above or below the water table at a time t. After an interval of time Δt the level will have recovered by an amount Δh. Then the flow may be approximated by using equation (1.6.21):

$$\frac{dQ}{dt} = \frac{k\rho g}{\mu}\left(h - \frac{\Delta h}{2}\right)A(a.d.s) \qquad 1.6.26$$

This equation becomes less of an approximation as the head interval Δh becomes small. For most situations Δh will be negligible when compared with h. A further approximation may, therefore, be made by ignoring Δh in equation (1.6.26).

The quantity of water seeping into the soil dQ as the level within the tube changes by Δh is:

$$dQ = - \pi a^2 \Delta h \qquad 1.6.27$$

Substitution of this equation into (1.6.26) leads to the following result:

$$- dt = \frac{\pi \mu a^2}{k \rho g \; A(a.d.s)} \frac{\Delta h}{h} \qquad 1.6.28$$

Integration of this equation between the time limits t_1 and t_2 and the potential head limits h_1 and h_2 yields the following:

$$t_2 - t_1 = \frac{\pi \mu a^2}{k \rho g \; A(a.d.s)} \; Ln \frac{h_1}{h_2} \qquad 1.6.29$$

It follows from equations (1.6.25) and (1.6.29) that

$$K = \frac{\pi a^2}{A(a.d.s)} \frac{Ln \frac{h_1}{h_2}}{t_2 - t_1} \qquad 1.6.30$$

Had the assumption not been made that Δh was negligible compared to h the following equation would have been derived:

$$- \int_{h_1}^{h_2} \frac{\Delta h}{h} = \frac{KA(a.d.s)}{\pi a^2} \left\{ \int dt - \int \frac{\Delta h}{2h} dt \right\}_{t_1}^{t_2} \qquad 1.6.31$$

In this equation the relationship between $\frac{\Delta h}{2h}$ and time is not known and, as a result, the last function cannot be integrated.

As the water level in the tube approaches equilibrium, i.e. $h \longrightarrow 0$, the last term may assume some significance. rendering the solution inaccurate.

The magnitude of the error may be assessed, however, by assuming a relationship between $\frac{\Delta h}{2h}$ and time. Assume that $\frac{\Delta h}{2h}$ is constant during the time period t_1 to t_2. It then follows that:

$$K = \frac{\frac{\pi a^2}{A(a.d.s)} \; Ln \frac{h_1}{h_2}}{\int_{t_1}^{t_2} \left(1 - \frac{\Delta h}{2h}\right) dt} \qquad 1.6.32$$

It is clear that the true value of the hydraulic conductivity will exceed the value obtained by ignoring the ratio $\frac{\Delta h}{2h}$. In order that the error should not be greater than 10%, the head interval Δh should be less than 0.2 h.

Equations (1.6.1) and (1.6.2) may be applied to a seepage tube where the cavity extends for some distance w below the tube, merely by including this extra variable in the shape factor term. This follows because the solution of Kirkham is obtained by summing the potential gradient at the water table surface, where the boundary conditions remain unaltered by the size or shape of the cavity.

1.6 (iv) The extension of Kirkham's method to anisotropic soils

Childs (1952) has shown that the solution of Kirkham may be applied to anisotropic soils. The hydraulic conductivities Kx, Ky and Kz on the three principal axes x, y and z may be related by the following equations:

$$\left(\frac{Kx}{Ky}\right)^{\frac{1}{2}} = \alpha \quad \text{and} \quad \left(\frac{Kx}{Kz}\right)^{\frac{1}{2}} = \beta \qquad 1.6.33$$

The co-ordinates describing the anisotropic soil may be mathematically expanded or contracted, so that they describe a theoretically isotropic soil. Let the new co-ordinate axes be described by x, λ and ε. The transformed axes λ and ε are related to the former by the following relationships:

$$\lambda = \alpha y \quad \text{and} \quad \varepsilon = \beta z \qquad 1.6.34$$

The isotropic value of conductivity K' is also related to the change in the co-ordinate axes thus:

$$K' = \frac{Kx}{\alpha\beta} \qquad 1.6.35$$

Equation (1.6.21) may be applied to this medium (for a tube with a cavity of length w in the anisotropic soil) as follows:

$$\frac{dQ}{dt} = K'A(a\alpha, d\beta, s\beta, w\beta)h \qquad 1.6.36$$

The shape factor depends to a much greater extent upon the dimensions of the cavity than the distances to the impermeable barrier s, or the water table d. When s and d are greatly in excess of the cavity dimensions, equation (1.6.36) may be simplified thus:

$$\frac{dQ}{dt} = K'A(a.\alpha, \beta w)h \qquad 1.6.37$$

The transformation of the co-ordinate axes alters the shape of the seepage tube and its surroundings. The seepage tube, which is cylindrical in the anisotropic soil, becomes elliptical in the transformation to isotropic conditions. The length of the tube will also be altered. In the isotropic soil only the x-axis remains unaltered. Equation (1.6.37) may be re-written to include the undisturbed terms:

$$\frac{dQ}{dt} = \frac{Kx}{\alpha\beta} A(a.\alpha.\beta w)h \qquad 1.6.38$$

This equation cannot be solved by one seepage tube experiment. If, however, the anisotropy is limited to the vertical z direction and the horizontal xy plane, i.e. $\alpha = 1$, a solution may be obtained.

Substituting the following continuity relationship:

$$dQ = -\pi a^2 \Delta h \qquad 1.6.39$$

into equation (1.6.38), and using the fact that $Kx = Ky = K_H$, it follows that:

$$-\pi a^2 \Delta h = \frac{K_H}{\beta} A(a.\beta w)h\ dt \qquad 1.6.40$$

Whence:

$$-\pi a^2 \int_{h_1}^{h_2} \frac{\Delta h}{h} = \frac{K_H}{\beta} A(a.\beta w) \int_{t_1}^{t_2} dt \qquad 1.6.41$$

Finally:

$$\pi a^2 \beta \, Ln \frac{h_1}{h_2} = K_H \, A(a.\beta w)(t_2 - t_1) \qquad 1.6.42$$

This equation may be partly solved by substituting values of h and t, obtained experimentally, in the expression. This yields an equation where β and K_H are unknowns.

The experiment may be reproduced in the same soil using a different cavity size (a'W) from which a similar equation to (1.6.42) may be obtained thus:

$$\pi a'^2 \beta \, Ln \frac{h_3}{h_4} = K_H \, A(a'\beta W)(t_4 - t_3) \qquad 1.6.43$$

A value of β may be deduced by solving equations (1.6.42) and (1.6.43) simultaneously thus:

$$a'(t_2 - t_1) A(a,\beta w) Ln \frac{h_3}{h_4} = a(t_4 - t_3) A(a'\beta W) Ln \frac{h_1}{h_2} \qquad 1.6.44$$

Values of β are inserted into this expression until equality of both sides is obtained. This value re-substituted back in equation (1.6.42) yields a value of K_H.

Now $\beta = \left(\frac{Kx}{Kz}\right)^{\frac{1}{2}}$ or changing terms $\left(\frac{K_H}{Kv}\right)^{\frac{1}{2}}$. It is therefore theoretically possible to determine the vertical and horizontal conductivities by the seepage tube method.

SECTION 1.7

The Shape Factor

1.7 (i) The determination of the shape factor

Consider equation (1.6.22)

$$A(a.d.s) = -\frac{1}{\phi_{(cav)} - \phi_{(wt)}} \int_{a}^{\infty} \left.\frac{\delta\phi}{\delta z} 2\pi r \, dr\right|_{z = d} \qquad 1.6.22$$

A(a.d.s) will have the dimensions of length regardless of the dimensions of ϕ. It will depend mainly upon the cavity dimensions rather than the distances to the water table or impermeable boundaries.

The implications of equation (1.6.22) are twofold. Firstly, it must be possible to obtain, by analysis, a theoretical derivation of A. Secondly, the idea of an analogue is suggested, in which the geometrical boundaries are the same, and in which it is possible to measure the parameters independantly.

Consider the first point. Kirkham tried to find a theoretical solution but, at that time (1945) no solution had been found. A solution has been developed, however, for the situation where the flow from or to the cavity approximates to the flow which would occur from a point source, or to a point sink.

The potential arising at a point from a sink or a source situated some distance r away from the point, and exclusive of constants due to the source or sink is:

$$\phi = \pm \frac{1}{r} \qquad 1.7.1$$

The potential arising at that point from a number of such sources or sinks is obtained by the sum of the respective potentials. The potential due to one source at a distance r_1 from the point and one sink at a distance r_2 from the point is:

$$\emptyset = \frac{1}{r_1} - \frac{1}{r_2} \qquad 1.7.2$$

The sign convention that a source is positive has been adopted. The diagram shown in fig. (1.7.1) approximates to this situation. The sphere, source S1, is supplied with water by a tube, the shape of which minimises the disturbance to the resulting flow pattern. The flow pattern which would be obtained by inserting a similar tube in the field is obtained, theoretically, by siting a similarly shaped sink S2 at a distance 2d from the source. It follows from fig. (1.7.1) that:

$$r_1 = (z^2 + r^2)^{\frac{1}{2}} \qquad 1.7.3$$

and

$$r_2 = (r^2 + (z - 2d)^2)^{\frac{1}{2}} \qquad 1.7.4$$

The shape factor, which in this case is $A(a.d.r_{sp})$, follows similar developments to those seen in equation (1.6.11) to (1.6.22) and leads to the equation:

$$A(a.d.r_{sp}) = \frac{1}{\frac{1}{r_{sp}} - \frac{1}{2d}} \int_a^\infty \frac{\delta\emptyset}{\delta z}\, 2\pi r.dr \quad z = d \qquad 1.7.5$$

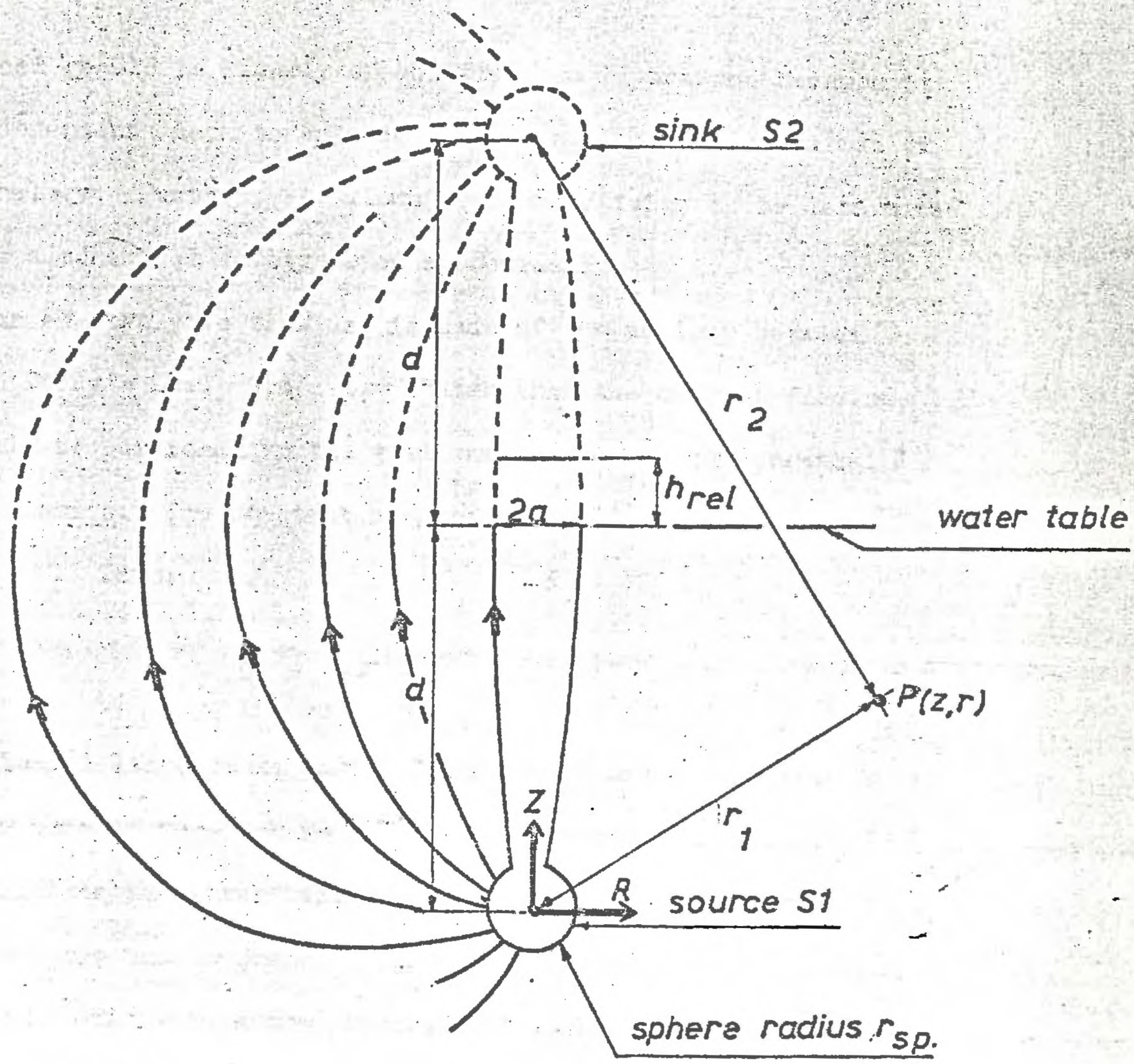
which is analogous to equation (1.6.22).

Substitution of equations (1.7.2), (1.7.3) and (1.7.4) in equation (1.7.5) leads to the following:

$$\int_a^\infty \frac{\delta\emptyset}{\delta z}\, 2\pi r.dr \quad z = d \;=\; \int_a^\infty \left((r^2 + z^2)^{-\frac{1}{2}} - (r^2 + (z - 2d)^2)^{-\frac{1}{2}} \right) 2\pi r.dr \quad z = d \qquad 1.7.6$$

Finally, it is found that:

$$A(a.d.r_{sp}) = \frac{4\pi\left(1 + \left(\frac{a}{d}\right)^2\right)^{-\frac{1}{2}}}{\frac{1}{r_{sp}} - \frac{1}{2d}} \qquad 1.7.7$$



Sketch of the relative dispositions of a source S1. and an imaginary sink S2. and part of the streamlines resulting from flow between them

Fig (1.7.1)

It is worth noting that when d is large, the shape factor approximates in the manner shown below:

$$A(a.d.r_{sp}) \rightarrow A(a.r_{sp}) = 4\pi r_{sp}$$

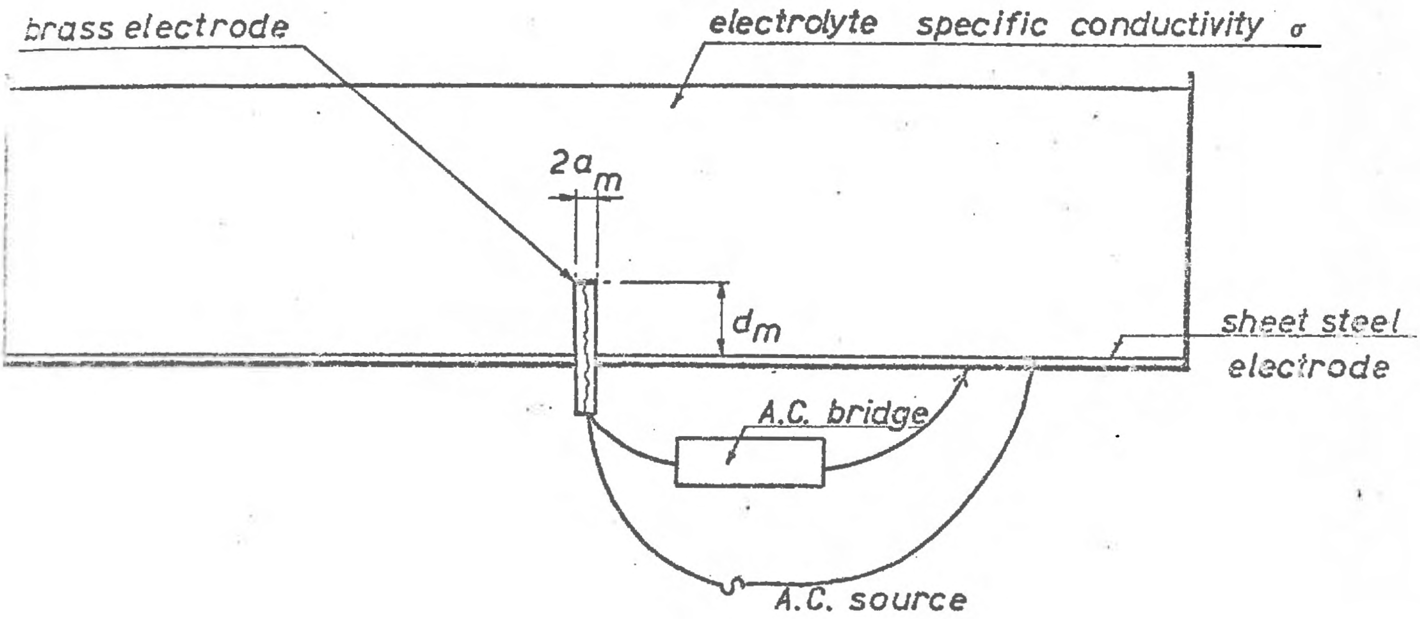
This analytical result is clearly of considerable importance because it allows an independant check to be made upon shape factors determined by electrical analogy. Further, it allows the conductivity to be determined in situ using a spherical cavity or an approximation to it.

In an electrolytic tank use is made of the analogy between Ohm's Law and Darcy's Law. Ohm's Law states that the current flowing I is directly proportional to the gradient of electrical potential V (voltage) at a point. The constant of proportionality is termed the conductivity . Thus:

$$I = -\sigma \frac{dV}{dz} \qquad 1.7.8$$

Consider a large tank, partly filled with an electrolyte solution. The base of the tank is an electrical conductor and is intended to simulate the water table. The electrolyte standing to some depth above the base represents the soil and water below the water table. The walls of the tank are required to retain the fluid but they are assumed to be sufficient distance from, say, the centre of the tank that their effect upon the flow pattern may be ignored. The simulated soil layer is considered to extend to infinity. Fig. (1.7.2) is a vertical cross section diagram of such a tank. An insulated tube protrudes through the tank base to a depth dm, and a brass cylindrical electrode is positioned on the top of this tube.

The unit represents a seepage tube installed into the soil below the water table, with a cavity at its lower end of zero length in this case. The metal tank base and the cylinder form part



Section on the Electrolytic
Tank

Fig (1.7.2)

of an A.C. electrical circuit. The current flow which results will have an identical pattern to the flow of water in the field. In considering the derivation of the necessary equations the simplifying assumption is made that the model has the field dimensions.

By analogy with equation (1.6.10) it may be written that:

$$V = d\phi + e \tag{1.7.9}$$

V is the voltage difference between the metal plate and the brass electrode. The equations are developed in a similar manner to those from (1.6.10) onwards. It is found that:

$$V = \frac{V_{(cav)} - V_{(wt)}}{\phi_{(cav)} - \phi_{(wt)}}\,\phi + V_{(cav)} - \frac{V_{(cav)} - V_{(wt)}}{\phi_{(cav)} - \phi_{(wt)}}\,\phi_{(cav)} \tag{1.7.10}$$

In the electrical sense there is of course no water table, however, the subscripts serve to remind us of the similitude.

The current i flowing from a small length δz of the sheet electrode is:

$$i = -\sigma \frac{\delta V}{\delta z} \tag{1.7.11}$$

so that the total current:

$$I_f = -\sigma \int_a^{\infty} \frac{\delta V}{\delta z}\, 2\pi r\, dr \tag{1.7.12}$$

Substitution of the value for $\frac{\delta V}{\delta z}$ obtained by differentiating equation (1.7.10) leads to the following:

$$I_f = -\sigma \frac{V_{(cav)} - V_{(wt)}}{\phi_{(cav)} - \phi_{(wt)}} \int_a^{\infty} \frac{\delta\phi}{\delta z}\, 2\pi r . dr \quad z = d \tag{1.7.13}$$

Consider now that the model is very much smaller. The ratio n between the model and field dimensions is:

$$n = \frac{a_m}{a_f} \qquad 1.7.14$$

a_m and a_f are the radii of the electrode size in the model and cavity size in the field. The current flowing in the model I_m would be similarly reduced

$$I_m = nI_f \qquad 1.7.15$$

Let the voltage of the brass electrode be V_1 and at the sheet steel plate V_2, instead of $V_{(cav)}$ or $V_{(wt)}$, then:

$$I_m = -\frac{a_m}{a_f}\sigma\frac{(V_2 - V_1)}{\phi_{(cav)} - \phi_{(wt)}}\int_a^{\infty}\frac{\delta\phi}{\delta z}\,2\pi r.dr \Big|_{z=d} \qquad 1.7.16$$

Substituting the result for the shape factor A(a.d.s), equation (1.6.22) into the above equation yields the following:

$$A(a.d.s) = \frac{a_f}{a_m}\frac{I_m}{\sigma(V_2 - V_1)} \qquad 1.7.17$$

The equation may be extended to apply to a cavity, of length w, by including the term in the expression which becomes A(a.d.s.w). Now V equals the product of I and R the resistance. Equation (1.7.17) reduces as a result to:

$$A(a.d.s) = \frac{a_f}{a_m}\frac{1}{\sigma R} \qquad 1.7.18$$

The shape factor is determined in an electrolytic tank by measuring the resistance between the brass electrode and the sheet steel electrode, and the specific conductivity of the electrolyte. The ratio between the model and the field is known. These values when substituted in equation (1.7.18) give the desired result.

1.7 (ii) Published values of the shape factor

The first values of the shape factor were published by Frevert and Kirkham (1948). They were obtained by using an electrolytic tank of some six feet diameter and with a maximum depth of electrolyte solution of twenty inches. The electrolyte was initially tap water but later copper sulphate solution was used. The electrode covering the tank floor, which simulated the water table, was constructed from 26 gauge copper and the cylindrical electrodes,representing the cavities at the end of the tube,were also constructed from copper. The values of shape factors obtained were published in a graphical form, and yielded the shape factor for cavities of zero length, having diameters ranging between 1" and 8". The effect of the impermeable boundary, simulated by the air/water interface, was assumed negligible, and the results do not therefore take this boundary into account. Of particular interest were the attempts to assess the disturbing effects caused by proximity to the cavity of stones or impermeable lenses; for the results showed that the effect was small whenever discontinuities were more distant from the cavity than its own diameter. This is of particular importance in a consideration of the conductivity of peat. The vegetation surface comprises highly compressible Sphagnum areas and resistant sedge hummocks which, it is inferred, will produce peats varying from highly conductive, on the one hand, to an almost impermeable condition on the other.

Luthin and Kirkham (1949) presented a graph, based upon results from the same tank, for a cavity with a length of four inches.

Youngs (1968) presented a comprehensive range of values of the shape factor which allow for the proximity of the impermeable boundary, and for most practical sizes of cylindrical cavity. These results were obtained in a small tank; however the limitation of size was compensated for by accurate measurements of the parameters involved. The values for the shape factor, determined by Youngs, are greater than those determined by Frevert and Kirkham by approximately 12%. Youngs concluded that the earlier results were due to a systematic error. In our own work, a large square tank, of similar dimensions to the tank used by Frevert and Kirkham, has been used but our results coincide approximately with those of Youngs. These results, together with a description of the electrolytic tank and the measurements made form the subject of an appendix to this thesis.

SECTION 1.8

The Formation of Mires and the Hydro-physical Properties of Peat

1.8 (i) The formation of mires

Romanov (1968) defines a mire as an area with abundant stagnant or slowly moving moisture in the upper soil horizons, overgrown by a specific vegetation with the predominance of species adapted to abundant wetting and to oxygen deficiency in the soil substrate. Peat accumulation is a characteristic feature, and the peat layer is sufficiently thick to prevent the roots of most plants from reaching the underlying mineral soil.

The work referred to in this thesis was done on a 'raised bog', called Dun Moss. A raised bog is a particular form of mire in which a more-or-less dome-shaped body of oligotrophic* peat has overgrown eutrophic* fen deposits, formed on a nearly horizontal mineral surface or in a rock depression, this overgrowth being accompanied by a continuous raising of the water table due to the capacity of the vegetation (in this country, especially of *Sphagnum*) to retain moisture. The surface of the mire is generally a mosaic of hummocks and hollows, and the junction between this surface and the surrounding mineral soils is marked by a peripheral system of natural drains (water tracks) in which eutrophic vegetation forms.

* Eutrophic: A stage in mire development characterised by the plant communities present. These include birch, alder, hypnoid mosses, reeds and various types of tall growing sedges. These plant communities are indicative of mineral rich conditions.

Oligotrophic: A stage in mire development characterised by such plants as *Sphagnum* moss, heaths, sedges and *Scheuzeria*. The communities are indicative of poor mineral status.

The initial eutrophic development depends to a large extent upon the topography of the underlying mineral soil. The oligotrophic development which spreads to the majority of the area of mire influence, depends upon innate regularities of development of the previous eutrophic stage. The continued deposition of peat perpetuates the conditions of impeded drainage which promoted the initial development. Viewed in section, the accumulated peat would have the appearance of a lens made up of many layers.

1.8 (ii) The formation and hydro-physical properties of peat

Peat is an accumulation of organic plant residue due to their incomplete decomposition. The partial decomposition occurs mainly as a result of aerobic decomposition in the upper layers. The submergence of the partially decomposed plants arrests the rate of reductive decomposition under anaerobic conditions.

Küster (1969) reports that Desulfovibrio, a sulphate reducing bacteria which produces hydrogen sulphide as a waste product, and methane forming bacteria were present in samples of peat.

Romanov (1968) reports that groundwater in peat deposits always contains dissolved gases and hydrocarbons in the lower layers. The solubility of the gases in water decreases with rising temperatures with the result that the gases are released as fine bubbles which greatly reduce the hydraulic conductivity of the peat.

This investigation is concerned mainly with the peat as it is found in the oligotrophic situation of Dun Moss. The bulk of this peat is formed from the partial decomposition of Sphagnum moss which is outstanding for the vast quantities of water it holds relative to its dry weight. The moss possesses ordinary chlorophyll bearing cells and in addition hyaline cells which are filled purely with water.

The apices of the stems from the complete, though porous, upper surface of large areas of Dun Moss. Rainwater falling upon the areas may percolate downwards between the individual stems of Sphagnum. It follows that the vertical hydraulic conductivity predominates at the surface. Eventually the moss is overgrown by new extensions to the plants and the older moss, beginning to decompose, seeks a flatter, more stable position. The stalks are now arranged horizontally, and Romanov (1968) reports that they frequently orient themselves in a direction parallel to that of the prevailing water movement. The horizontal conductivity now begins to predominate. Tissue maceration continues with the aid of aerobic bacteria. The resulting consolidation of the moss increases firstly, the intra-cellular water content, quoted as 50% of the total moisture content for surface peats by Romanov (1968), and secondly, the area of surface per unit weight of dry material (specific surface).

Volarovitch and Churaev (1968) report values of the specific surface as high as 25,000 $cm.^2/gm.$ for highly decomposed peat, however, Romanov reports a lower figure, 4,000 $cm.^2/gm.$ for similarly decomposed peat. They also state that peat is a multi-component polydispersed system. It comprises the hydrophobic sols and the hydrophilic semi-colloids. Peat such as is found in the oligotrophic stage of mire development is recognised as having a particularly high percentage of hydrophilic humin matter, and that the hydrophilicity increases with humification, having an optimum high at the middle ranges. A photo-micrograph of peat reveals a net-like structure of plant cell remains.

The pores between, and in the cells, are slit shaped and these slits are, in general laid horizontally. The implication is that anisotropy occurs, though there is little evidence from other sources to support this.

The total porosity is quoted as being as high as 0.9 by Romanov (1968). The effective porosity is lower than this, quoted as 0.5 by Volarovitch and Churaev (1963).

In a gel like medium, such as peat, consideration needs to be taken of the interaction existing between water and the solid. Volarovitch and Churaev record the fact that as much as 30% of the total water content of a peat, in the middle stages of humification, was physically bound. Similarly, Romanov reports a figure of 11% for a peat in the later stages of humification. Pokrouskii is quoted by Romanov as reporting that 'water in fairly decomposed peat may possess anomolous properties. A type of water exhibiting peculiar stability due to its reaction with solid particles'.

This fluid may exhibit shear strength and may require the establishment of a threshold gradient before flow commences. Osmotic effects, between the water associated with the hydrophilic colloids and that outside, causing swelling of the gel, have also been noted. The colloidal gel increases with decomposition which at the same time breaks down the cell walls releasing entrapped water.

In conclusion, the complexities of the material, and some of the recorded interactions between peat and water may be expected to complicate the estimation of the hydraulic conductivity. If realistic values are to be obtained then care must be taken to preserve the natural <u>in situ</u> conditions.

SECTION 2

THE HYDROLOGY OF DUN MOSS

SECTION 2.1

Dun Moss

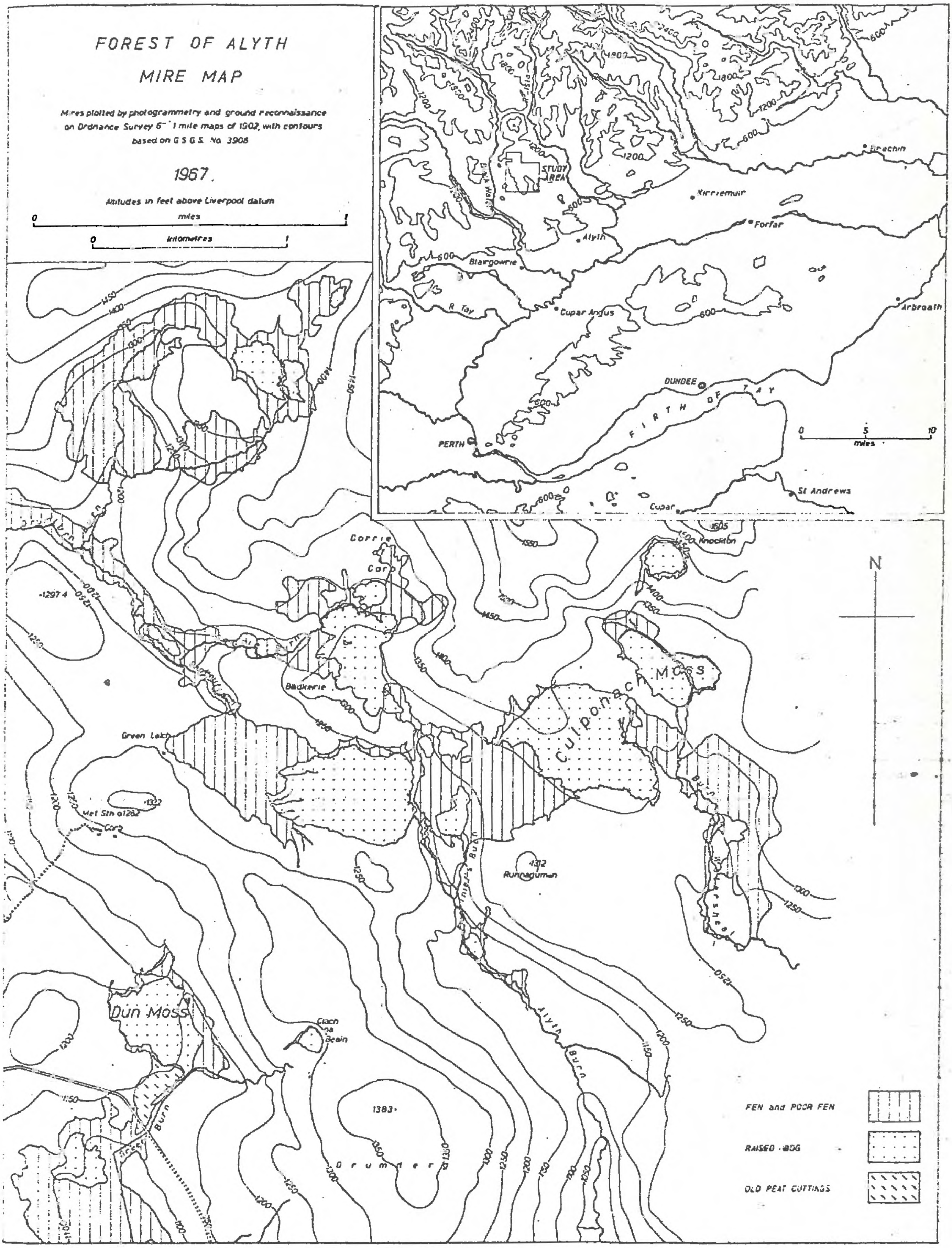
2.1 (i) Introduction

The inset portion of fig. (2.1.1) shows the general position of the study area. The main map of the study area shows the occurrence of peat areas which are catagorised as: fen and poor fen areas where eutrophic conditions predominate; raised bog where oligotrophic conditions predominate and old peat cuttings.

Dun Moss, upon which the investigations were carried out, is typical of the raised bogs in the locality, it is also the most accessible. It has unfortunately been interfered with by man, for: peat has been removed for fuel, the growing vegetation has been partly burnt and attempts have been made to drain it; however these disturbances do not appear to have altered the character of the raised bog significantly, for Sphagnum moss still flourishes across large areas and as far as it is possible to tell peat is still being deposited.

Plate (2.1.1) is a panoramic view of Dun Moss. The view, taken from the lower slopes of Clach na Boain (see fig. 2.1.1), shows the dome shape, characteristic of a raised moss. The bog lies across a saddle between two hills and the view is taken looking across this saddle. Water drains from the northern (right hand side of the photograph) and the southern ends of the bog. The two burns into which drainage is received both form part of the Tay river system.

A conspicuous drainage ditch, trending S.E.-N.W. has been cut across the middle of the bog. This ditch which is shown in plate (2.1.2) was approximately four feet deep initially, but it is now partly filled in with Sphagnum moss. Drainage ditches were also excavated along various lengths of the perimeter of the bog and one



FOREST OF ALYTH
MIRE MAP
Mires plotted by photogrammetry and ground reconnaissance on Ordnance Survey 6" 1 mile maps of 1902, with contours based on G S G S. No. 3906
1967.
Altitudes in feet above Liverpool datum
miles
kilometres
Brechin
Kirriemuir
Forfar
Alyth
Blairgowrie
STUDY AREA
R Tay
Cupar Angus
Arbroath
DUNDEE
FIRTH OF TAY
PERTH
St Andrews
Cupar
Corrie
Corb
Knockton
Blackerie
Green Loch
Culponach Moss
Met Stn
Runnagumen
Dun Moss
Clach na Bein
Alyth Burn
Drumder
N
FEN and POOR FEN
RAISED BOG
OLD PEAT CUTTINGS

Fig (2.1.1)

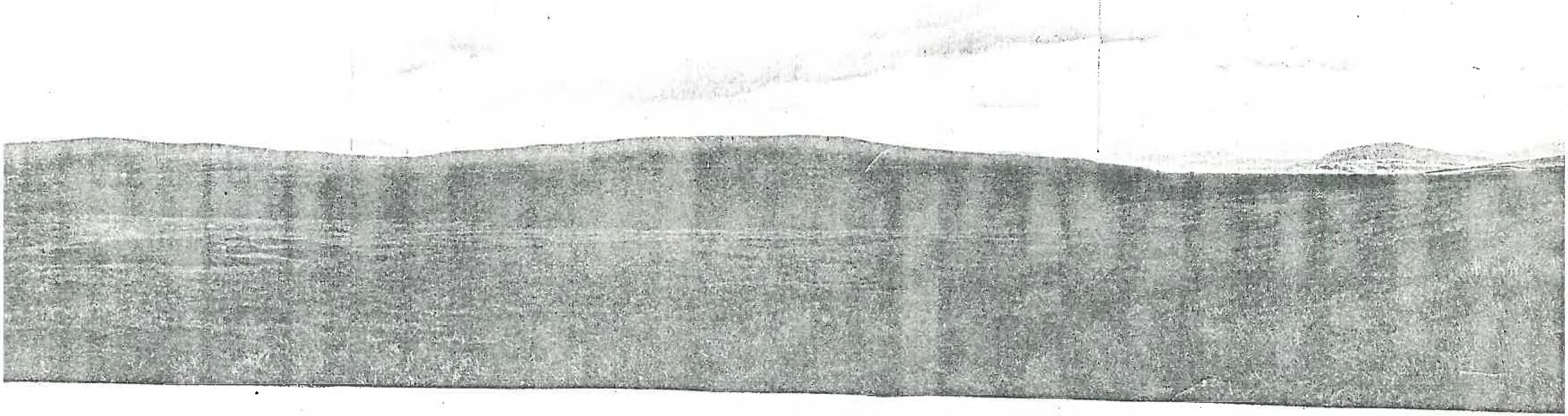


Plate (2.1.1)
View of Dun Moss from the East

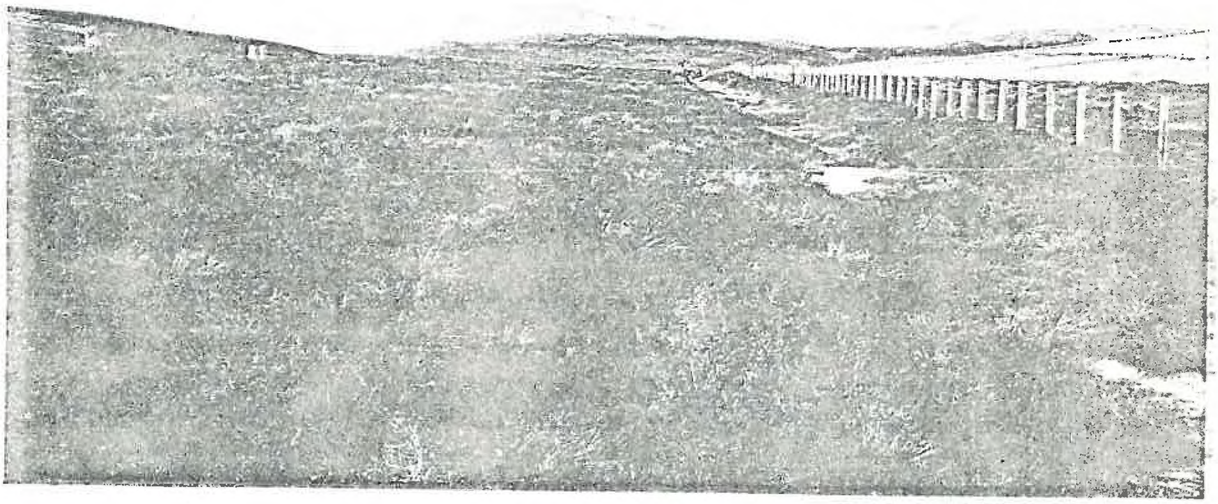


Plate (2.1.2)

Dun Moss Showing Central Drainage Track

of these may be seen in plate (3.2.1) page. 142. It seems reasonable to suppose that the central ditch was excavated to receive water from an intended system of lateral ditches which were never dug, and that the peripheral ditches were excavated to prevent surface runoff from the mineral soil entering the bog. The ditches still function as drains, and the central drainage track discharges that water which it removes from the bog at its northern and southern ends. The region of influence of this ditch does not seem to be great extending of the order of fifteen yards either side of the ditch. The water drains from the northern end of the ditch into a eutrophic fen area. Plate (2.1.3) is a view of the fen taken in the direction of the flow of water which is derived from a spring. The contrasting vegetation between the fen and the raised bog, is clearly shown in the photograph.

2.1 (ii) The geological features of the Dun Moss area

The geological features of the area determine the condition in which the mire developed. From the inset to fig. (2.1.1) it can be seen that the study area lies in the Grampian foothills. These are composed of a confusing mixture of metamorphic Dalradian rocks, thrust up above the Devonian sediments of the Lowlands to the south of the Highland Boundary Fault. This runs across the country from N.E. to S.W. and, according to Allan (1928), outcrops some three miles S.E. of the study area.

It is known from tunnelling operations in connection with a water undertaking, some five miles to the N.E., that the rocks north of the fault zone are rendered porous by extensive fractures and fissuring, which may strongly influence the geo-hydrology of the area.

Plate (2.1.3)

Fen Track At Northern End of Dun Moss

The geological situation is obscured by the processes of glaciation, more particularly by the processes of erosion and deposition occurring during the last glaciation.

Charlesworth (1955) presents an analysis of the ice movements in Scotland during the last glaciation. Generally speaking, the ice that influenced the study area moved down S.E. from the Cairngorm massif. The body of ice was constrained to follow the glens formed by previous geological processes. Such glacier movements occurred down the Glens of the Blackwater and Isla. The pressure of ice to the south in the broad valley of Strathmore created relieving lobes on the higher ground between these glaciated glens, thus flows of ice occurred from one glen to the other, possibly across the study area. In the vicinity of Dun Moss there may have been movement across the saddle and down the valley to the North West, and possibly down the valley to the South East as well. Whatever the sequence, when the ice began to melt and recede, it left considerable quantities of drift on the study area. By largely obscuring the underlying solid rock, these deposits have had the largest single influence upon the geo-hydrology of the area. The drift lies like a thick blanket over the area, its depth uncertain. Its effect has probably been the creation of a barrier to the efficient penetration of rain water.

Springs occur from various places, on the drift lying on the hills bounding Dun Moss. They are recognizable by the effect they have upon the vegetation, for they give rise to patches of close-cropped grass in areas of otherwise drab vegetation. There is no reason to suppose that the springs are limited to the hillside; they may well occur in the area overgrown by the peat. Equally, there may be areas under the bog where water from the peat may escape. This almost certainly happens in the peat bog situated in the Corrie of Corb, fig. (2.1.1) where an extremely deep erosion hole has been formed in the centre of the bog. Another interesting topographical feature is the presence, on the eastern flanks of the saddle containing the bog, of deeply incised natural gullies one of these may be seen in the background of plate (2.1.3). The depth of these gullies is a testimony to the quantities of water which have flowed down them in former times.

2.1(iii) Principal features of the vegetation of Dun Moss

The principal features are shown on the plan of Dun Moss, fig. (2.1.2). The surface of the raised bog is a mosaic of alternating hummocks and hollows. The vegetation is varied. Sphagnum moss, of several species, occupies the largest area. The hollows being wet, support almost wholly Sphagnum, and a type commonly met with is S. papillosum. The hummocks again are often formed of Sphagnum, S. capillaceum being one example. This drier zone permits the growth of heath plants, lichens and sometimes grass. The tussocks of sedges, i.e. Eriophorum vaginatum are a notable feature. These tussocks are dense and hard in contrast to the spongy nature of the Sphagnum communities. The central drainage track and the ditch near the western edge are occupied by very wet, loosely-packed Sphagna, especially S. papillosum and, in the wettest places, S. cuspidatum. There are very few open water pools, even in the wettest weather. The wetter parts of the mire margin are occupied by various fen communities. Carex rostrata occupies the northern fen between K_1 and K_3, and occurs at intervals along the eastern edge. It is usually found with its stem bases standing in water which is often visibly moving. Elsewhere the marginal water tracks are marked by clumps of Juncus effusus, associated with Sphagnum recurvum and Polytrichum commune. Where wet hollows have been left after peat cutting, tussocks of Eriophorum vaginatum grow amongst Sphagna, especially S. recurvum.

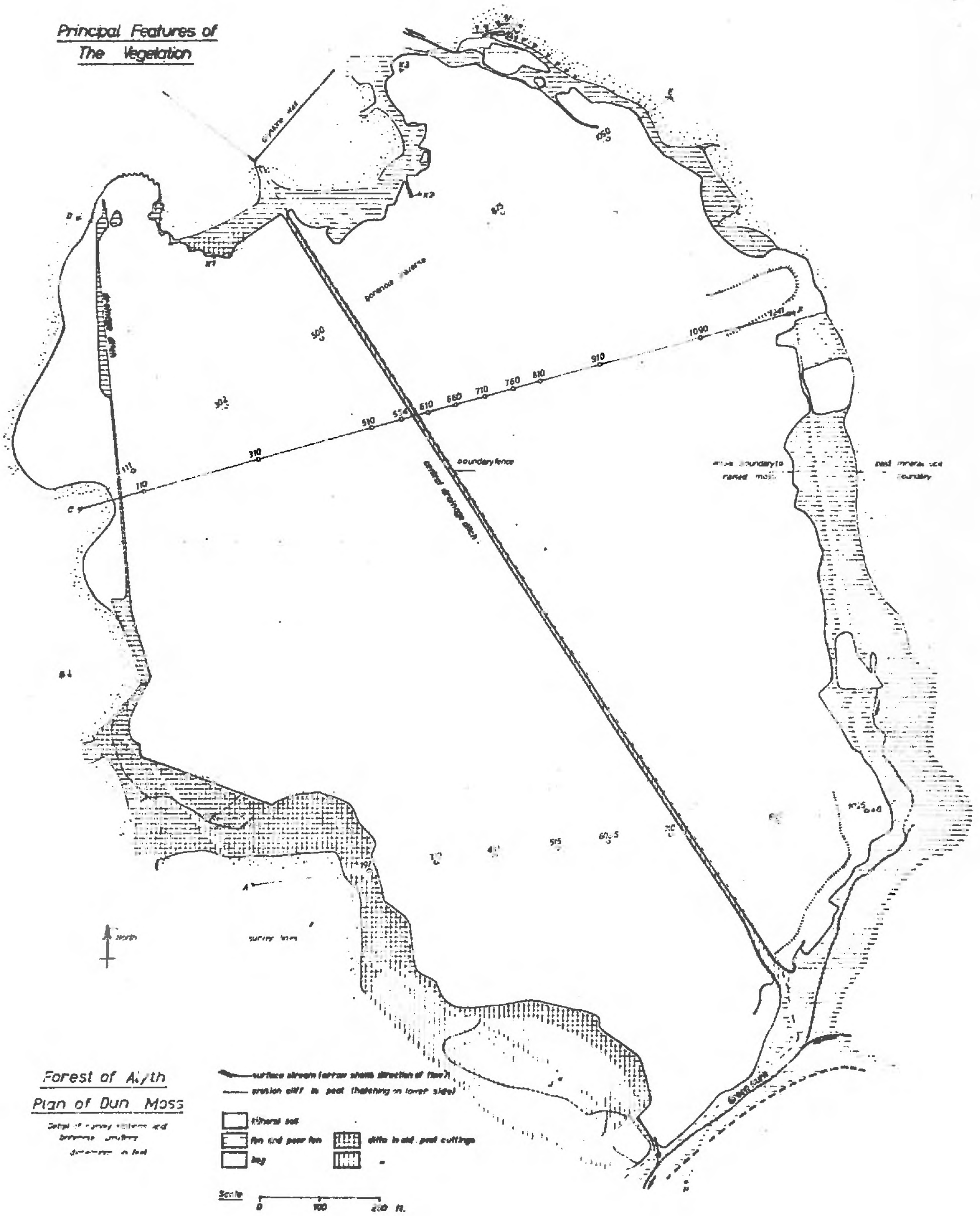

Principal Features of
The Vegetation
boundary fence
central drainage ditch
drainage ditch
North
Forest of Alyth
Plan of Dun Moss
Scale
0
100
200 ft.

Fig (2.1.2)

SECTION 2.2

The Survey of Dun Moss and its Environs

2.2 (i) The survey of the surface topography

It was felt that a survey was necessary for the following reasons:

(a) To locate the study area via co-ordinates relative to the ordnance survey.

(b) To lay down a control for subsequent operations in the vicinity, e.g. the establishment of lines of boreholes between individual survey stations.

(c) To delineate the boundaries of the peat and the major topographical and vegetational features.

(d) To obtain contours so that slopes etc. could be obtained.

A series of survey stations (see fig. (2.1.2) page 84) were with one exception established upon the mineral soil surrounding Dun Moss. Together they formed a closed traverse, the total length of which was 1,510 metres. Their relative dispositions were based upon sighting distances and the relative levels between them. Each station was located by using electricity conduit hammered well into the ground to avoid interference by grazing animals. The positions of the stations were determined during 1967 by chaining and by a theodolite survey (20" vernier). The results of this survey were adjusted according to Bowditch's rule. The levels of the stations, relative to the ordnance datum, were determined firstly, by determining their levels relative to each other (Watts autoset) and secondly, by relating these levels to the level of the nearest bench mark (O.S. ref. No. 169547) some 1.6 Km. from instrument station A.

The survey, required to relate the station levels to the bench mark, was done during the winter of 1967/68 at a time when the visibility was extremely clear, and the ground was frozen solid. The instrumental error involved in surveying a distance of 1.6 Km. was minimised by using equal backsights and foresights. A new fixed point of reference was established on a large stone adjacent to Dun Moss.

The positions and relative levels of posts, driven into the peat, on three sections across Dun Moss A-G, C-E and C-F were also determined during the summer of 1967. Difficulties of atmospheric refraction limited the maximum sighting distance across the bog to about 500 ft. It was found that a slight breeze helped eliminate the refraction. The outline of the boundary between the peat and the mineral soil was surveyed, using offsets from the imaginary lines joining adjacent stations. The lines were measured using a 100 ft. chain and the offsets were measured using a flexible tape. A stick, with which the soil could be probed, was found useful in those positions, where a difficulty existed of recognizing the boundary between the peat and the mineral soil.

The detailing of the steep northerly end of Dun Moss was facilitated by the establishment of three subsidiary stations K_1, K_2 and K_3. The information available was used to draw a plan of Dun Moss, fig. (2.1.2).

A contour map of Dun Moss was made as a record of its surface topography, and as a basis for assessing the pattern and direction of water movement near the surface, fig. (2.2.1). Ideally, measurements should be made of the level of the water table. It was decided that this was impractical in view of the great number of positions at which levels would need to be obtained, and also in view of the transient nature of the water table. It was felt that the water

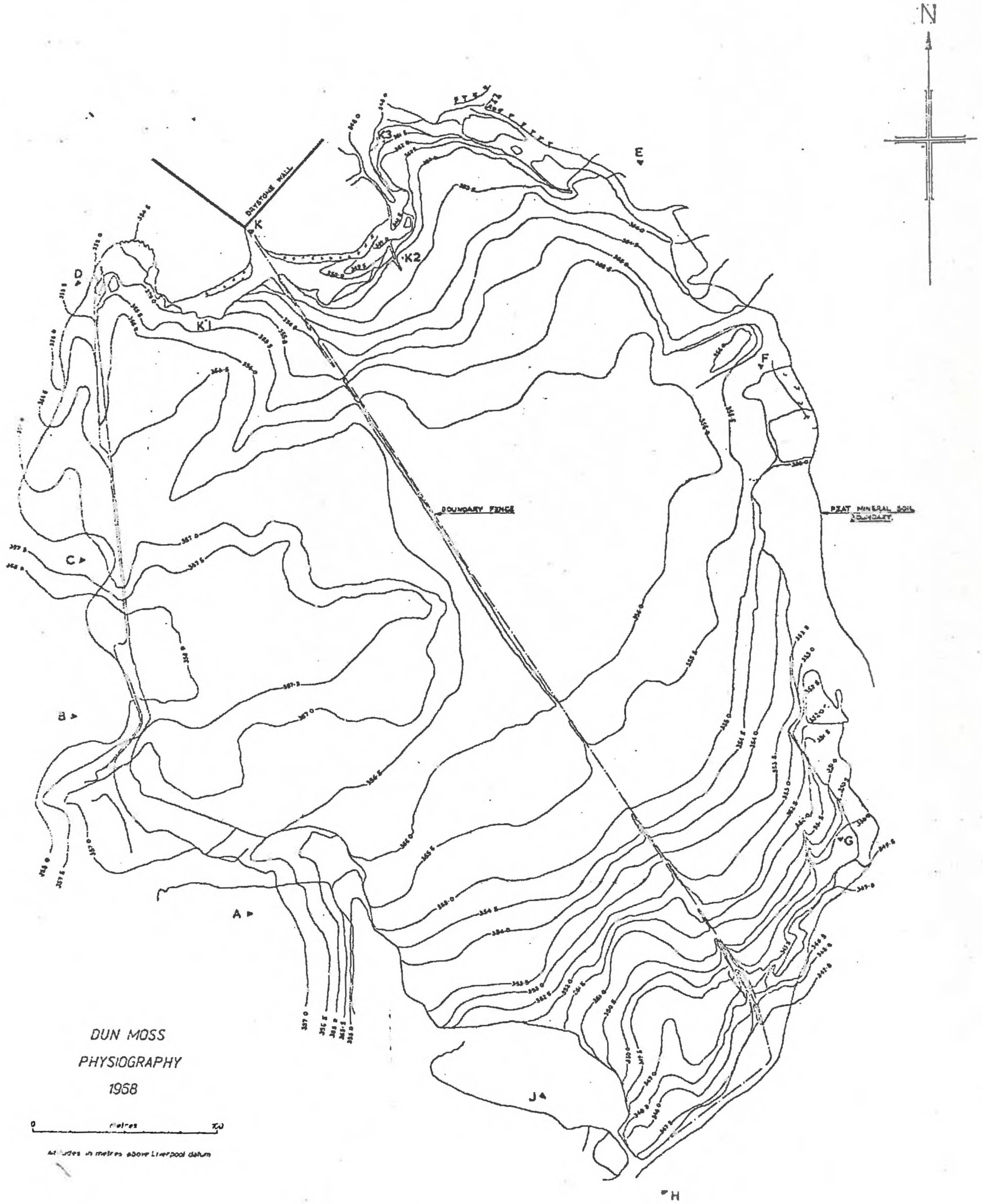


Fig (2.2.1)

Contour Map of Dun Moss

table at Dun Moss was sufficiently near the surface for the levels, and slopes of that surface, to give an approximate idea of the levels and slopes of the water table surface. The surface of Dun Moss is, however, a mosaic of hummocks and hollows; as a result, it was decided to measure the levels to the lower points of the hollows.

The survey work was carried out in the summer of 1968 using a 20" Watts microptic theodolite and two metric staffs. The survey instrument was centred on the individual stations established upon the mineral soil, and tacheometric readings were taken at points on rays radiating out from the instrument. The rays between adjacent stations overlapped to give a full coverage. The chosen contour interval was 50 cm . and the positions were chosen with this in mind. Experience of surveying upon the bog showed that a staffman could position himself on hummocks or tussocks, meanwhile holding the staff on the position in a hollow without affecting the level of the staff.

The contour map shows a number of features. Firstly, the central drainage track lies almost exactly on the longitudinal axis of the bog, secondly, the contours suggest the likely pattern of surface drainage on the bog. They indicate that the central area is very flat and this is in fact the wettest part. The water drains radially away from the east side of the central ditch, but on the west side of the channel the situation is different, for water moves to the north and south, and also in a direction from B towards the central drain. The north and south ends of the bog have steep gradients, and they are drier than the centre because the water table tends to be lower in these areas relative to the bog surface, an effect possibly accentuated in places by the proximity of erosional cliffs.

2.2 (ii) The stratigraphic survey of Dun Moss

It was thought that a stratigraphic survey of Dun Moss was necessary for a number of different reasons among with were the following:

(a) A peat bog exists in three simensions; therefore a survey would not be complete without some knowledge of the underground topography.

(b) Subsequent experiments, involving peat at depth, would benefit by a knowledge of the underlying mineral soil topography and the composition, texture and dispositions of the various peat types.

Initially it was thought that the structure of the bog would be best revealed by a series of stratigraphic sections across the width of the mire in the east-west direction. Accordingly the lines C-E, C-F and A-G were chain-surveyed and numbered posts were set out at intervals of 60 m. or 100ft. The levels to the basesof the posts were established by levelling with the autoset from instrument stations established on the mineral soil, adjacent to C, E, F and beyond G.

A 'Hiller peat borer' was used in the investigations. This borer is shown in plate (2.2.1). It is constructed from two concentric stainless steel cylinders both of which are slotted. The cylinders are fitted one inside the other in such a manner that they may be rotated independantly thus opening or closing a slot. In operation, the closed corer, attached to extension rods, is lowered to the desired depth in the peat. The chamber is then opened by rotating the extension rod. Continued rotation in the same direction causes peat to be cut by a blade disposed along the appropriate slot length, and guided into the chamber. The chamber is then closed by rotating in

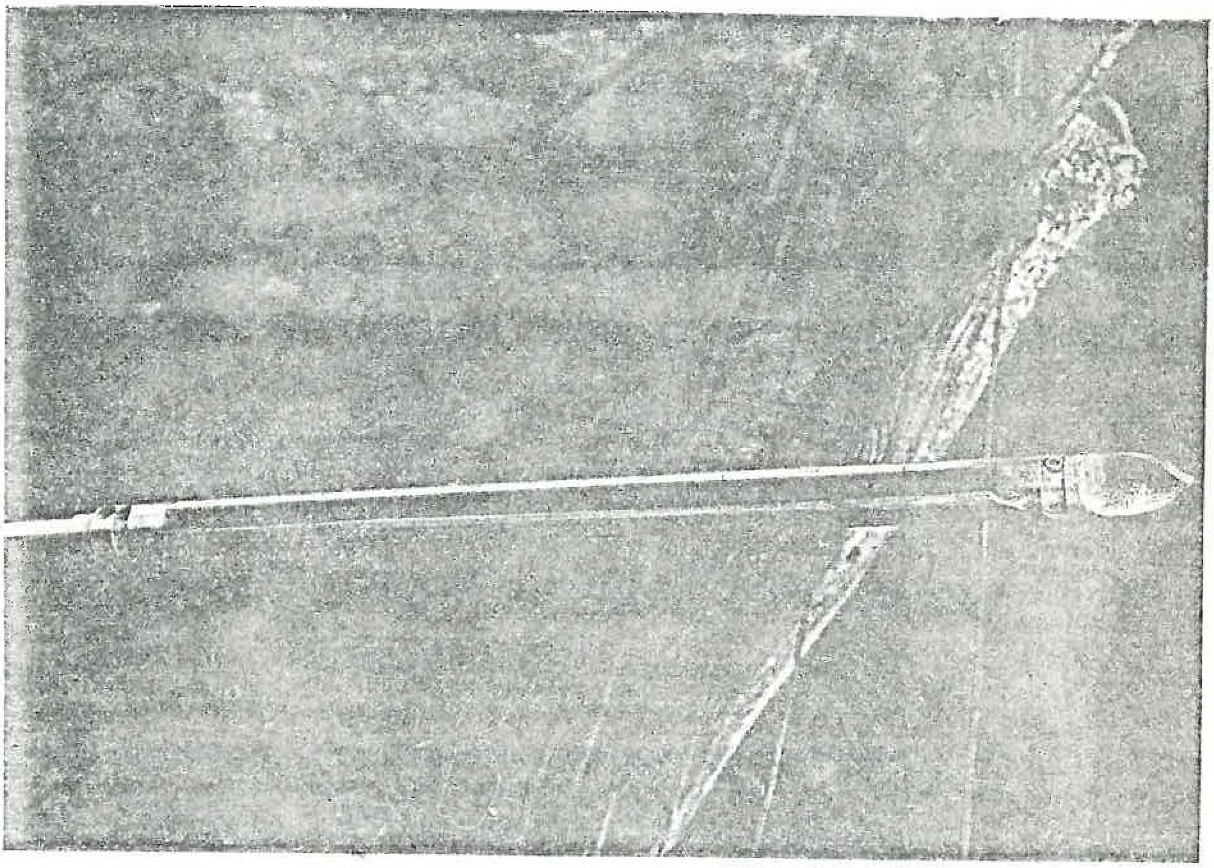


Plate (2.2.1)

Hiller Peat Borer

the opposite direction after which it may be removed in order that the peat may be examined.

The 50 cm. long column of peat, contained within the chamber, was first of all subjected to a visual inspection to determine colour, constituent materials and discontinuities in the profile. The identification of plants such as Sphagnum, Calluna or Ericaceae and sedges was only difficult for highly decomposed peats. The peat was then classified according to the humification scale of von Post and Granlund (1926). A translation of which is shown on the following page. The degrees of humification shown are based upon rheological properties of peat, and upon visual estimations of the proportion of remaining undecomposed material in a sample. The scale was not at all difficult to apply and it was sometimes possible to classify peats between two humification degrees.

The information resulting from the investigations during the summer of 1967 was used to draw up sectional diagrams for the lines C-E, C-F and A-G. Fig. (2.2.2) is* the diagram obtained for the line C-F. At a number of boreholes, investigations of the stratigraphy were limited to the lower layers of peat only in an attempt to obtain more details of the position of a deep buried channel. The channel was located on all three section lines and this information revealed that it sloped downwards towards the north. As a result of these investigations it was decided to investigate the stratigraphy of the north-south axis of the bog, line K_2H. This was done in the summer of 1969.

The mineral soil below the peat was generally light grey in colour and it had a smooth consistency. The layer of peat above the mineral soil was, in a number of situations black and amorphous. It contained Potamogeton seeds and other remains associated with lacustrine

* Fig.(2.2.2) is contained in the pocket sewn into the back cover.

Ten-point humification scale

(von Post and Granlund)

When indicating the humification of peat, a 10-grade scale is employed.

H_1: No humification, peat free from ooze; when squeezed (in the hand) limpid, colourless water is produced.

H_2: A mere touch of humification only and peat almost free from ooze, when squeezed produces limpid but tawny water.

H_3: Little humification, or peat slightly muddy ((emits turbid water when squeezed). No peat-substance passes between the fingers). The residue is not pulpy.

H_4: Slightly more humification or peat increasingly muddy. When squeezed emits (strongly) sludgy water. The residue somewhat pulpy.

H_5: Moderate humification or peat rather extensively muddy. The structure of growth fully distinct but somewhat veiled. When squeezed some peat (substance) passes between the fingers in addition to intensely muddy water. Residue sludgy.

H_6: Moderate humification or peat rather extensively muddy. The structure of growth not fully distinct. When squeezed maximum $\frac{1}{3}$ of the peat passes between the fingers. The residue is more sludgy but indicates more distinct structure of growth than the unsqueezed peat.

H_7: Strong humification or peat extensively sludgy (in which the structure of growth is still visible). When squeezed about one half of the peat-substance passes between the fingers. If water is emitted, this is gruel like and strongly dark coloured.

H_8: Very strong humification or peat intensively sludgy with structure of growth hardly visible. When squeezed about $\frac{2}{3}$ of the peat substance passes between the fingers. Gruel-like water may be produced. The residue consists mainly of more resistant root-threads etc.

H_9: Nearly complete humification or peat almost entirely ooze-like (in which hardly any structure of growth is indicated). When squeezed, nearly the whole peat substance passes like a homogeneous gruel between the fingers.

H_{10}: Humification and oozification complete. No structure of growth can be discerned. When squeezed the whole bulk of the peat passes between the fingers without any emission of water.

When the extent of humification is situated some where between two grades (or intervals), its indication can be given as say H_{2-3}, H_{7-8} etc.

—oOo—

conditions. Along the north-south axis of the bog Phragmites peat was found above the black lake deposits, and above this the bark and wood of birch trees. Outside the main channel area the lowermost deposits were birch (Betula) and sedge (Carex), then across the whole of the bog, the birch gave way to associations of Sphagnum mosses, cotton-sedge (Eriophorum vaginatum) and heaths (mainly Calluna) which continued to the surface.

The accuracy of the recorded depth of any peat layer was limited. Normally the depths were referred to a fixed horizontal bar in the vicinity of the borehole. Inevitably, however, with two people standing nearby, alterations to the level of the bar must have occurred. It was felt that any investigations of hydraulic conductivity in the peat should take place some distance away from these areas of disturbance near the boreholes. As a result, an accuracy of ± 5 cms. in the absolute level of the borehole record would suffice for positioning of tubes etc. The borehole results have an estimated accuracy within this range.

SECTION 2.3

The Instrumentation of Dun Moss

2.3 (i) Introduction

The instruments were chosen to fulfil the aim of providing information on the general hydrology of the mire as a background to studies of, initially, hydraulic conductivity of peat. It was decided to measure precipitation on the area, the evapo-transpiration from the vegetation cover and the response of the water in the surface layers and at depth to these and other related phenomena. The measurement of the absolute runoff from the peat catchment was considered desirable as a secondary objective, but in the event it was not practical. The positions of all instruments in the Dun Moss area are shown on fig. (2.3.1).

2.3 (ii) The measurement of precipitation

The situation, types of rain gauge used, and the period for which measurements are available are detailed in the following table.

Position	Grid Reference	Elevation ft.	Type of Instrument Used	Frequency of Readings	Period During which Results Available
Corb steading	No.166569	1300	Mk. 2 Standard 5" gauge	Weekly	Oct 1967-Oct 1969
			Mk. 2a Octapent 5" gauge	Monthly	Oct 1969-
Dun Moss (adjacent to hut.)	No.166561	1150	Mk. 2 Standard 5" gauge	Weekly	Oct 1969-
			"	"	"
			Tipping Bucket	Continuous	April 1969
Dun Moss (adjacent inst.) Station C)	No.165558	1150	Mk. 2a Octapent 5" gauge	Monthly	Aug 1968-Oct 1969
			"	Weekly	Oct 1969

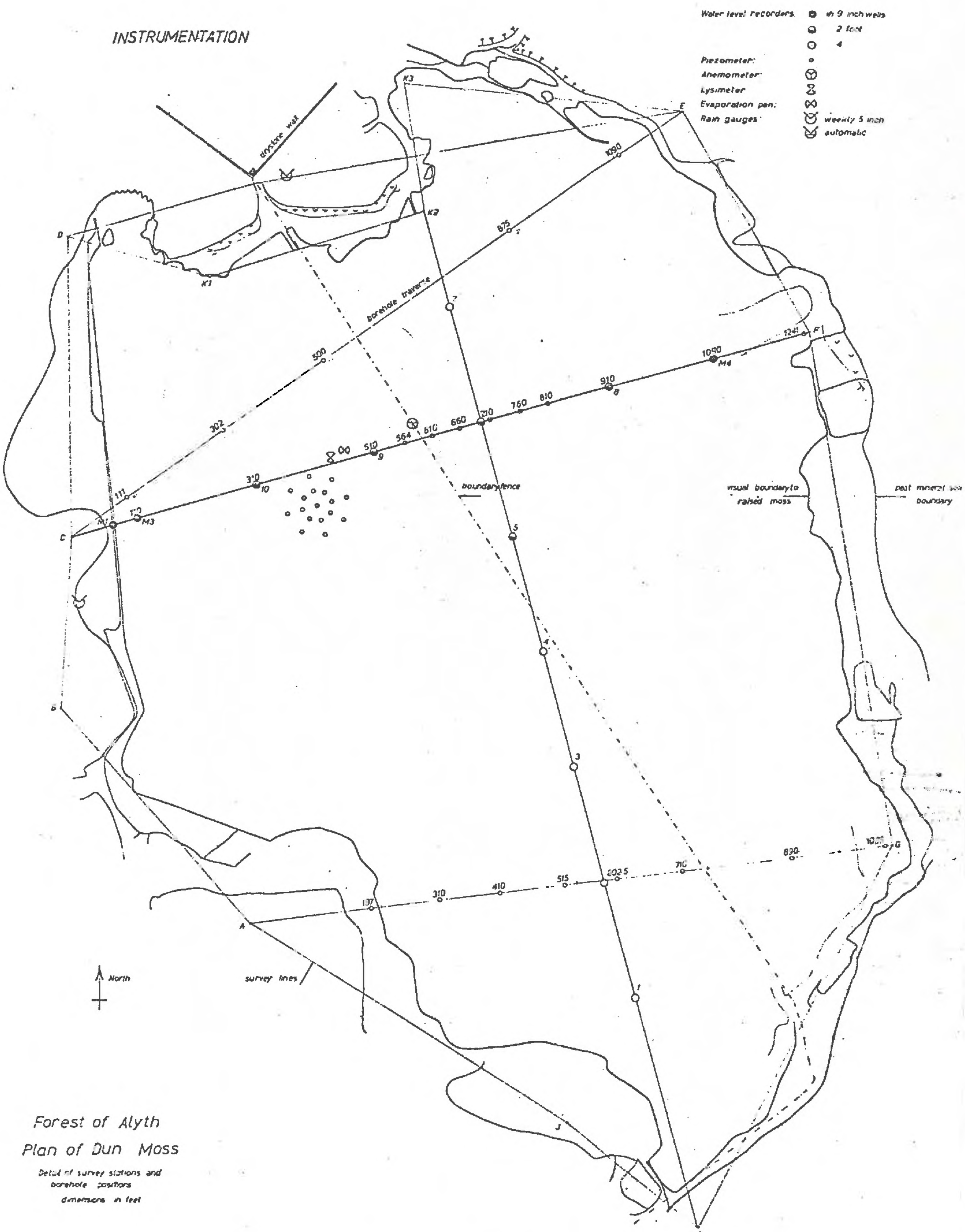

INSTRUMENTATION
Water level recorders
in 9 inch wells
2 foot
4
Piezometer:
Anemometer:
Lysimeter
Evaporation pan:
Rain gauges:
weekly 5 inch
automatic
drystone wall
borehole traverse
boundary fence
visual boundary to raised moss
peat mineral soil boundary
survey lines
North
K1
K2
K3
D
E
F
C
B
A
J
H
G
M3
M4
111
302
500
875
1090
310
510
564
610
660
710
760
810
910
1090
1241
137
410
515
2025
830
Forest of Alyth
Plan of Dun Moss
Detail of survey stations and
borehole positions
dimensions in feet

Fig (2.3.1)

The tipping bucket rain gauge (Artech plastics Ltd.) has a catchment area of 750 cm^2 and tips for every 0.02 cms. rainfall, see foreground of plate (2.3.1). The incidence of rainfall is recorded upon an Evershed and Vignoles multi-pen event recorder which is situated inside the hut seen in the background of the photograph. The 9'0" long x 6'0" wide hut had been erected on the mineral soil adjacent to the northern perimeter of Dun Moss during the autumn of 1968.

2.3 (iii) The measurement of evapo-transpiration

It was decided to install instruments to measure the evapo-transpiration directly from a section of the bog, and evaporation from an open water surface to act as a comparitive standard. Accordingly a peat filled lysimeter, plate (2.3.2) and an evaporation pan, plate (2.3.3) were installed on the bog adjacent to the G-F line, fig. (2.3.1). The peat filled lysimeter was installed during the summer of 1968. It was constructed from a fibre glass tank. The depth was 66 cms. and the plan dimensions to the outside of the rim were 88 cms. and 78.5 cms. Its depth was chosen to avoid interference with the living roots of plants which would be placed in it.

A volume of peat, similar to the dimensions of the tank, was cut out of the bog, placed inside the tank which was then manouvered back into the excavation. In the period following, the lysimeter was kept well supplied with water to allow the vegetation to recover. A small amount of deterioration occurred to Sphagnum moss immediately adjacent to a small well which had been excavated at one edge of the tank. The moss was replaced by fresh growth which quickly became established.

The water level in the lysimeter is maintained approximately constant using a 6 gallon supply reservoir and a 6 gallon draining reservoir. The supply is regulated using a carburettor float chamber

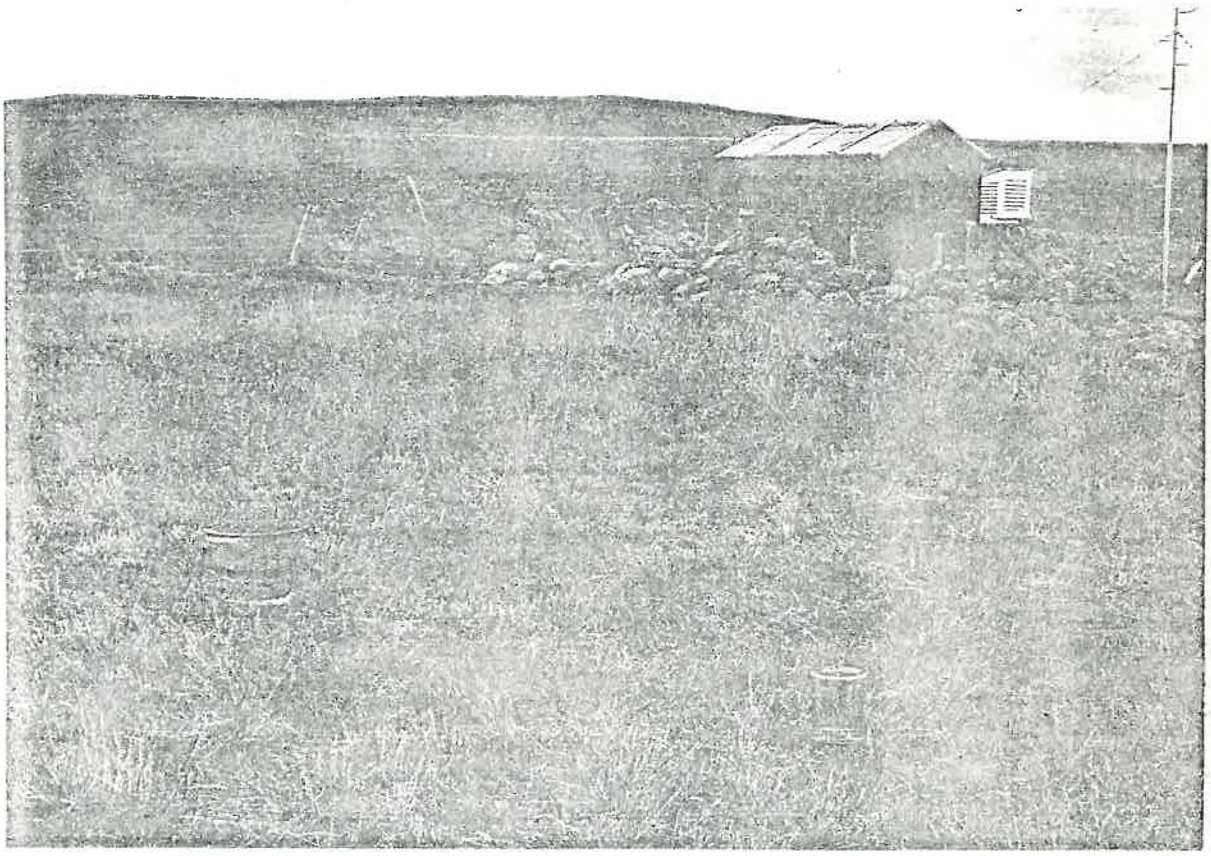


Plate (2.3.1)

Raingauges Adjacent to the Hut at
Dun Moss

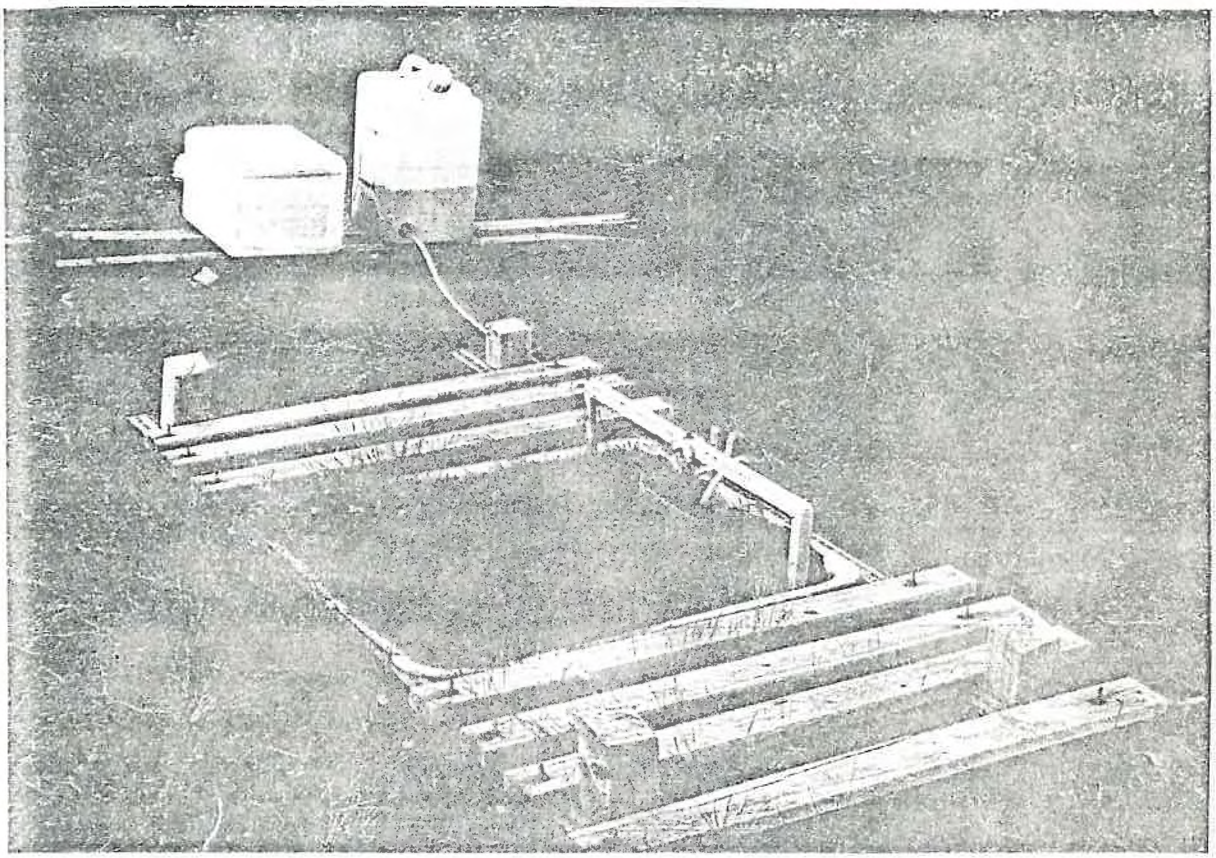


Plate (2.3.2)

Peat Filled Lysimeter

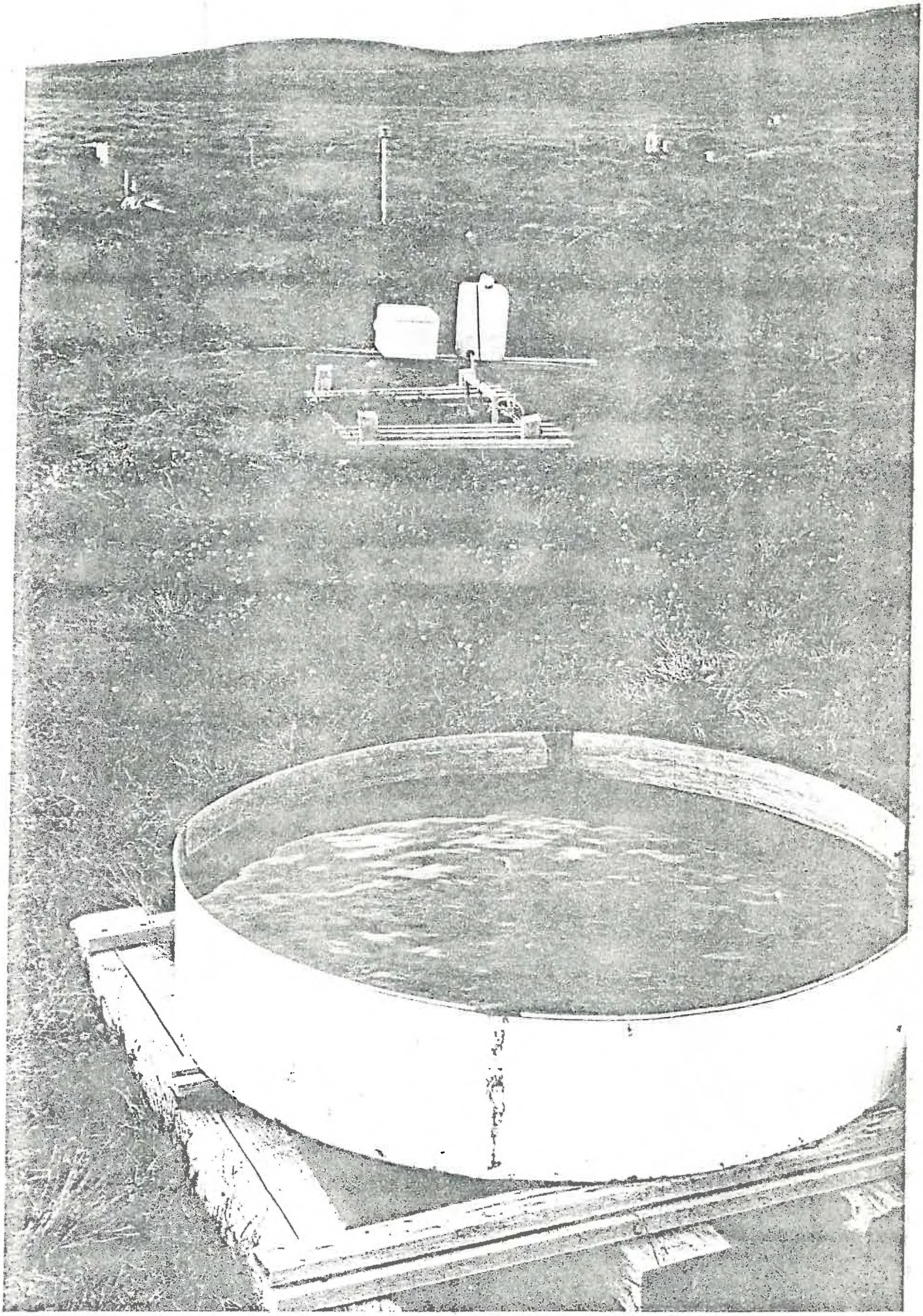


Plate (2.3.3)

Evaporation Pan

and the drainage using a funnel. Both these devices are supported at fixed levels in the well excavated in the peat at one edge of the tank. The difference in level between the funnel and the float chamber is approximately 1 cm.

The lysimeter subsided following installation, as a result it was raised by about 5 cm. and supported upon an angle framework, itself supported by four posts driven four feet into the peat. Weekly measurements have been made since March 1969 of the amount of water supplied and, by means of a pump attached to the submerged drainage reservoir, of the amount drained.

The evapotranspiration is computed from the observed values of flux of water into and out of the lysimeter. A number of features combine to make the computation more difficult than it need be. Firstly, the tank has a flat rim; as a result it is difficult to absolutely define its catchment area. It was assumed that the area was bounded by an imaginary line running half way across the flat part of this rim. The tank itself tapers in slightly with depth, and any fluctuation in the water level in the tank must depend upon the plan areas of the tank at the water levels. The presence of an open well for the supply seems unavoidable, it will however complicate the interpretation of computed results. A number of difficulties also arose in the operation of the lysimeter. Firstly, the carburettor float chamber supply occasionally became blocked; though the incidence of this was less than anticipated. The blocking seemed to be caused by a fungal growth rather than by particles in the water supplied. The drainage reservoir was only sufficient to hold the overflow from 1.2" rainfall but, at certain seasons, this was inadequate. The internal diameter of the tube (9 m.m.) used to remove this water was too small, with the result, that the removal of 6 gallons of water became a lengthy and tedious operation.

An U.S. Bureau of weather class A land pan was installed during the Spring of 1969. The pan was four feet diameter and ten inches deep. It was supported horizontally, with its rim 17 inches above the level of the bog, by a wooden framework. The water level in the pan has been recorded weekly since March 1969.

2.3 (iv) The measurement of the water table response

The positions of the thirteen water level recorders used are shown on fig. (2.3.1). They were installed during the Summer of 1968. Ten are described as 'Barnesbury Recorders' which are manufactured by Negretti and Zambra, plate (2.3.4). The remaining three are a modification of a Munro float recorder, plate (2.3.5).

Changes in water level in a nine inch diameter well results, in the Negretti recorders, in a change in the volume of a rubber balloon suspended below the water surface of the well. The surface mounted recorder, connected to the balloon by a copper capillary, measures the volume changes and records them on a disc chart calibrated in terms of the change in water levels. The disc chart is rotated once weekly by a clockwork mechanism which requires weekly rewinding. A Munro instrument records the fluctuations in level of a float, sited in the well, on a drum chart. These charts have a nine inch range and are also rotated weekly and require weekly rewinding.

Five of the Barnesbury recorders have a range of two feet; accordingly, the balloons were sited on the floor of wells which were 2' 3" deep, and the remaining five have a range of four feet, these were sited at the base of wells 4' 3" deep. The floats on the Munro instruments operate in wells which are 2' 3" deep. The wells were excavated using a draining spade after which they were lined with galvanised wire mesh cages to prevent their deformation. The recorders

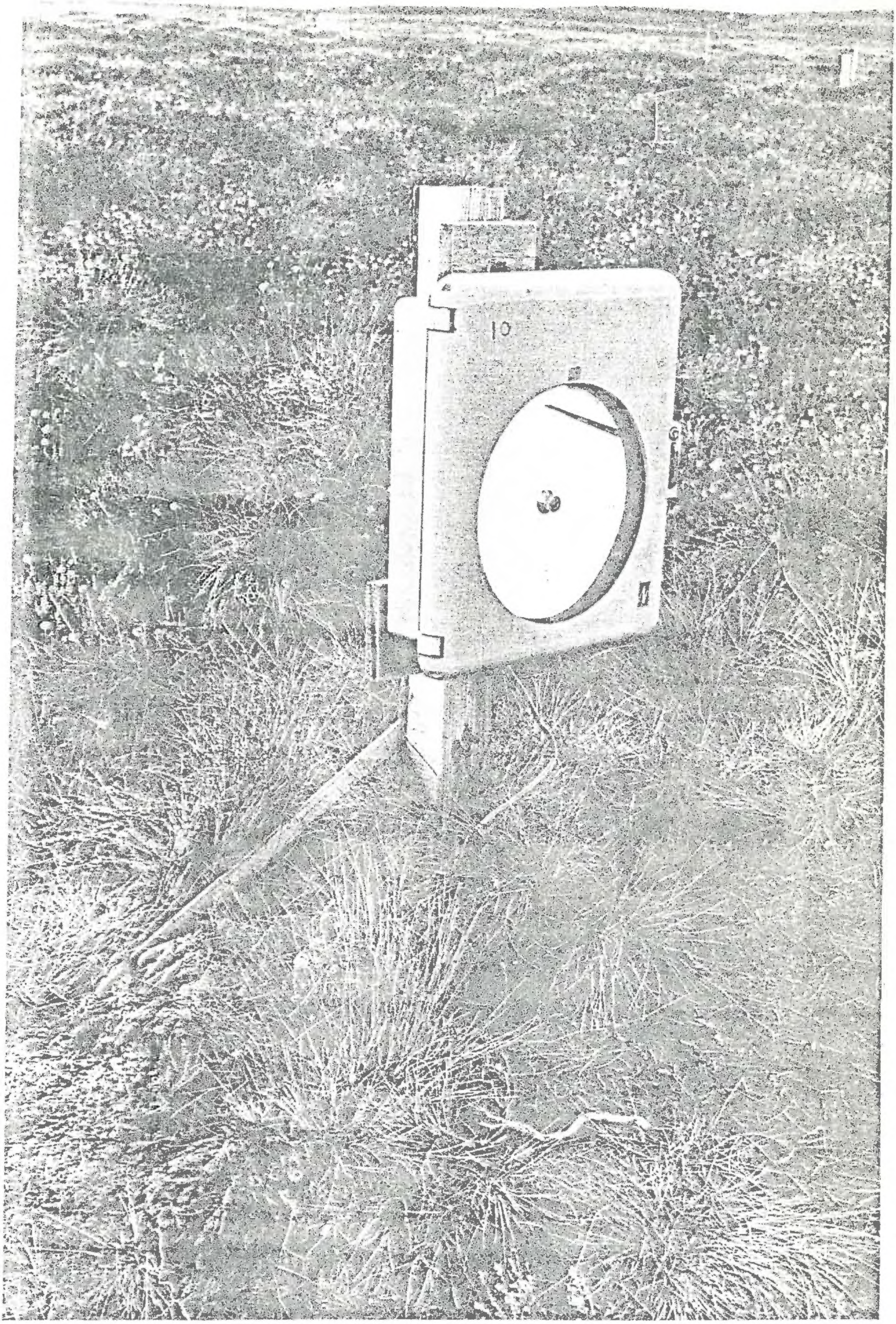


Plate (2.3.4)

Negretti and Zambra Water Level Recorder

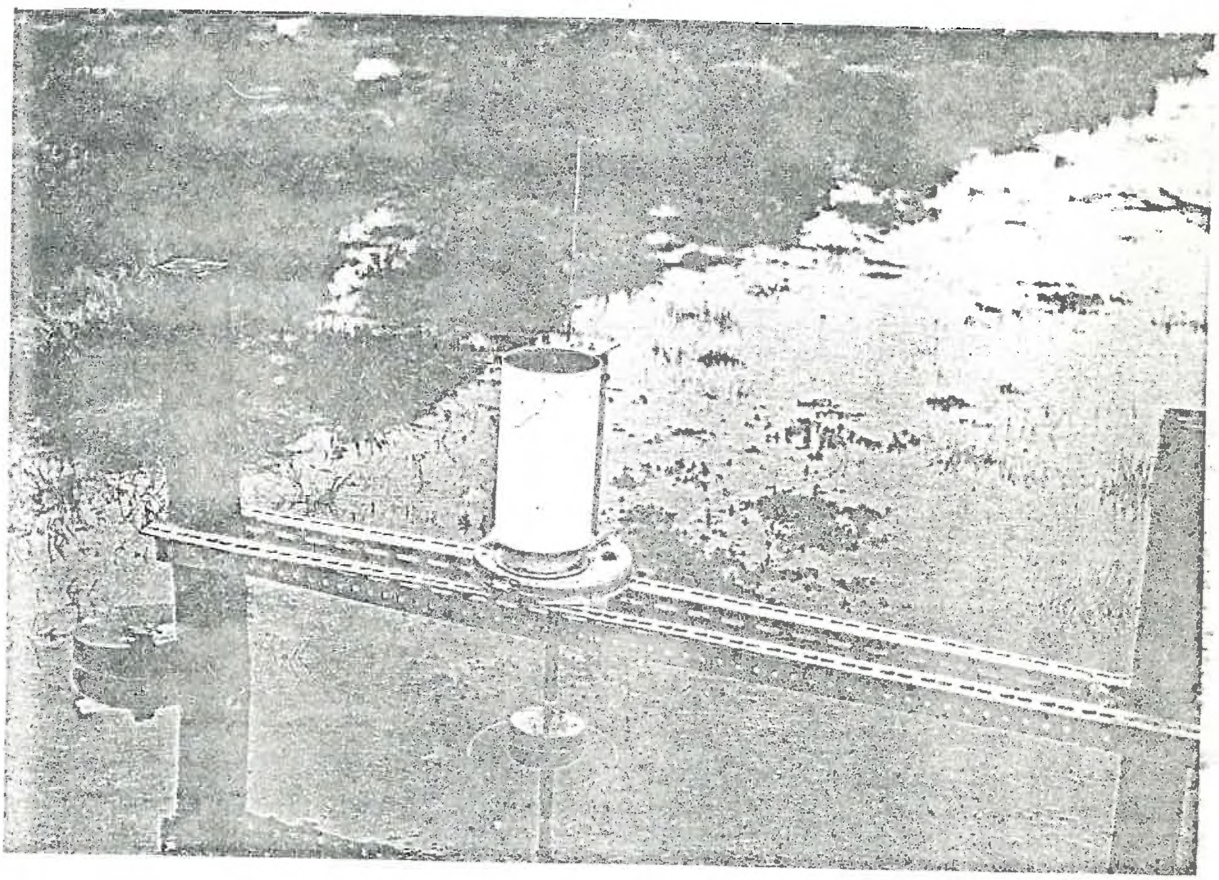


Plate (2.3.5)

Munro Water Level Recorder C090 F
in Sphagnum infilled ditch

are supported by fence posts driven well into the peat close to the wells.

The Negretti recorder housings are not completely weatherproof and a lot of trouble was experienced with the recorder charts which used to crumple up in damp weather. A number of solutions were tried before it was realised that the trouble was caused by slight condensation occurring on the stationary backplate which supported the rotating chart. This retarded the chart but failed to retard the clock mechanism which therefore screwed up the paper and spoilt the record by deflecting the pen. The final solution required the removal of the backing plate altogether, and the provision of an aluminium plate similar in dimensions to the chart but which rotates with the chart. Also, the pen mechanisms were not designed for windy conditions, though a new cartridge pen is now available, and these recorders are more difficult to install and maintain than the simpler drum recorders. It is suspected that mechanical friction and the low torque available combine to make the response of the Negretti recorder more delayed and less sensitive than that of the float-actuated Munro instruments. They have, however, one very great advantage which allowed for a most important modification to be carried out; for the response of the water level, in an open well, depends upon:the initial response of a water table surrounding it, the differential head levels between the two and some hydraulic conductance parameter, so that the response in the well may not reflect the true water table response. The simplest solution is to curtail the amount of reservoir storage in the well, in the limit reducing it to zero. (A well of zero diameter, effectively.) After consideration it was decided to backfill the wells with peat. It was not necessary to fill the whole well up but only that part experiencing the seasonal fluctuations. In August 1969 a wire mat was suspended 18" to 24" below the rim of each of the wells, and the space above each was filled with peat. The charts, removed following this modification detailed a more immediate response, by water levels to rainfall, than had hitherto been observed.

2.3 (v) Piezometric measurements

It was decided to seek to determine the actual vertical groundwater seepage in the peat. The initial concept envisaged the selection of small and, if possible, uniform areas of the mire which would be instrumented with fairly heavy densities of piezometers at various depths. The potentials which would be obtained would in themselves be of great interest and would permit the seepage to be computed, provided reliable values of hydraulic conductivity were available. It was hoped that the piezometers would enable a number of questions about the peat bog to be answered: firstly, what was the magnitude and direction of the vertical groundwater flux; secondly, what relation existed between the water table in the mire and in the mineral soil adjacent and below it; thirdly, what seepage relation existed between the lateral fen areas and the peat bog.

Time did not permit for more than the first of these questions to be examined. From the stratigraphical investigations it was known that the area between C 310F and C 510F was uniform. The lower soil surface was approximately level and various peat strata were recognizable. The area chosen is also shown on fig. (2.3.1). Part of the area had been burnt over about 1960 but it had recovered and showed an abundant Sphagnum growth. 16 positions were then set out on the circumference of concentric circles 100ft. and 50ft. diameter, and one at the centre of the circles. At each position three 1/4" I.D. aluminium alloy piezometers were installed terminating 11'6", 7'6" and 3'6" below the surface of the bog and one 1" dia. slotted aluminium pipe which terminated 18" below the bog surface. The resulting water level in the latter was held to correspond to the water table level.

In order that the water levels in the pipes could be read easily a reduced pressure manometric system was constructed, plate (2.3.6). The glass manifolds each had three stems connected to the piezometers by short plastic tubes. The fourth stem was connected to a longer piece of tubing, the lower end of which was immersed below the water level in the 1" dia. slotted tube. The whole system was continuous and connected to the atmosphere through a glass valve. A suitable reduction of pressure in the system caused the water to rise into the glass stems where their differential heads could readily be measured. The manometers were only fitted at the centre position and on the eight positions on the outer circumference.

During the installation of a piezometer, a ram rod was used to stiffen the pipe and also to prevent peat from entering the lower end. After installation it was observed that the response time of the water levels in glass stems, to changes of the water table level, was slow. The ramrod was again employed to create 3" long cavities below the piezometers. Twisted lengths of steel wool were then guided down the pipes into the cavities. Subsequently the response time of the water levels to an imposed displacement of about 1" was generally not longer than fifteen minutes.

The manometers were modified at a later date, since a system made entirely from glass was not ideal for the treatment received in their installation. In the final design the manifolds were made from copper, with stems no longer than 1". These were attached to the piezometers by longer plastic tubes which themselves replaced the manometric stems. The glass valve was retained but its forshortened glass stem was connected to the copper head by a short length of the plastic pipe. This final solution was very robust.

Rycroft. Part 3

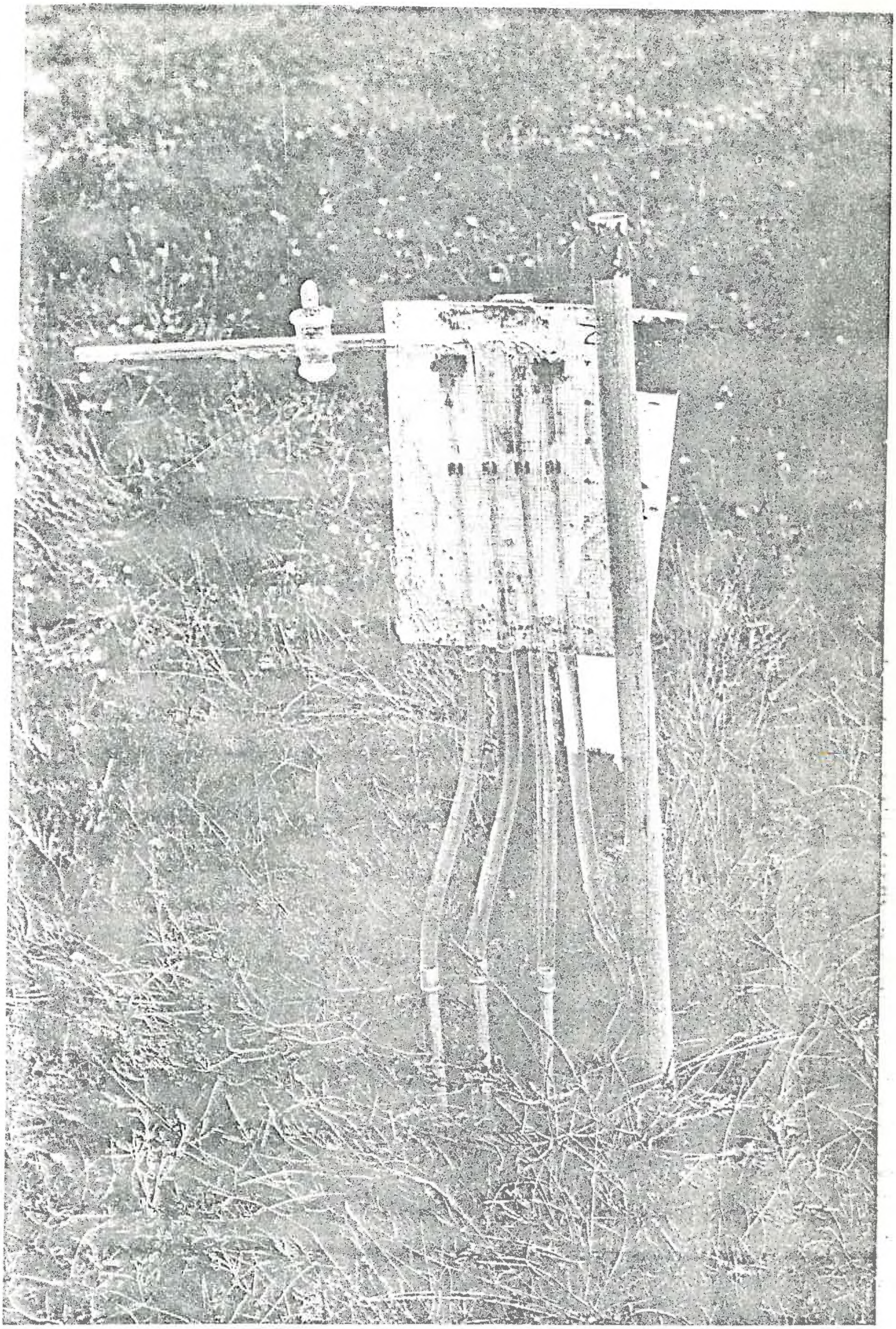


Plate (2.3.6)

Piezometric Installation

2.3(vi) The measurement of temperature and wind

A Negretti and Zambra 'Mersteel' type hygrometer was used to measure and record continuously the wet and dry bulb temperatures of two thermometers housed in a Stevenson screen,their range was from -10^{0}C to +40^{0}C.The Stevenson screen was sited a short distance from the hut at Dun Moss and the recorder was situated inside the hut.The paper disc charts were changed weekly.

The run of wind was measured with a Munro cup contact anemometer situated twenty feet above the ground on a guyed 2" dia. steel mast. The instrument operates by rotating a worm gear which momentarily closes a mercury switch for every 0.05 mile of wind.The number of contacts was recorded by the Evershed and Vignoles event recorder.This recorder operates at a roll chart speed of 2" per hour.It was found that the initial frequency of contacts was excessive;therefore a stepping relay was introduced into the circuit with the result that the deflections of the pen on the chart occurred for every0.5 mile of wind run.A readable record of average wind speeds up to 40 m.p.h. was then produced.The current was supplied by a unit of four 6 volt dry batteries connected in series.These,under Optimum conditions,last longer than three months.