The Regional Effect

of

Water Table Lowering

in the Durban Area

Thesis submitted in partial fulfilment of the requirements for the degree of Master of Science in the Department of Civil Engineering, University of Natal, by Brian Thomas Baxter.

16th July, 1973

I hereby certify that this research is the result of my own investigation which has not already been accepted in substance for any degree and is not being concurrently submitted in candidature for any other degree.

Brian Thomas Baxter.

I hereby certify that the above statement is correct.

Supervisor.

C O N T E N T S

S U M M A R Y

This thesis takes the form of a general approach to the possibility of major de-watering, and its effects, in the Durban area. In the light of the available information and resources an indication of the magnitude of the problem is given but the determination of absolute values is not possible.

An isohypal map of the area is developed and used in conjunction with a water balance to assess the recharge to the Durban City centre.

The drawdown for a hypothetical excavation is determined using a calculated average coefficient of permeability and the quantity of water pumped from the excavation is compared to the recharge.

The consequences of overpumping are considered with respect to the inflow of salt water from the sea and settlements of the soil.

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a = empirical function of I.

A = cross sectional area of standpipe (cm^2).

b_1, b_2 = half long and short side length respectively of rectangular group of wells.

β = K/E_o

B = Beaufort wind force.

B_o = Bowen's ratio.

γ = constant in wet and dry bulb psychrometer.

c = $\dfrac{1{,}5 \times C_{kd}}{P_o}$

C_{kd} = Point resistance of Dutch cone (lb/sq.in).

d = standpipe diameter (cm).

d_c = depth at which C_{kd} is taken.

D = diameter of intake sample (cm).

D_{10} = effective grain size, 10% finer.

Δ = slope of saturation pressure curve at air temp Ta.

Δp = change in effective overburden pressure P_o.

ΔM = change in moisture content and storage.

$\frac{dH_s}{dx}$ = hydraulic gradient.

e = void ratio.

e_a = saturation vapour pressure at air temp.

e_d = saturation vapour pressure at dewpoint (mm.Hg.)

e_s = saturation vapour pressure at evaporation surface.

e_z = saturation vapour pressure at height Z.

E = actual evapotranspiration

E_a = drying power of the air.

E_o = potential evaporation from open water surface (mm/day).

E_T = potential evapotranspiration from a grassed surface.

f_D = correction factor for precipitation.

F = well shape and size factor.

F_c = sum of monthly consumptive use factors.

G = groundwater recharge.

G_k = correction factor for partial penetration of well into aquifer.

h = active head (cm).

h_c = head at centre of well group.

h_d = head at any point in drawdown ($Hs-h_d$) equation.

h_f = head of fresh water above sea level at a point on the freshwater/sea water interface.

ho = head at time $t = o$.

h_t = head at time t.

hw = head at well.

H = heat budget.

H_c = thickness of soil layer in settlement equation.

H_s = saturated thickness of aquifer.

$H_{1,2,3}$ etc. = thickness of layer 1, 2, 3, etc.

I = heat index.

i = hydraulic gradient.

j = monthly heat index = $\left(\frac{T_M}{5}\right)^{1,514}$

k = coefficient of permeability (cm/sec).

k_a = average coefficient of permeability (cm/sec).

k_h = horizontal coefficient of permeability (cm/sec).

k_v = vertical coefficient of permeability (cm/sec).

$k_{1,2,3}$ etc. = coefficient of permeability of layer 1,2,3, etc.

K = convective heat transfer from a surface

K_B = empirical constant (Blaney & Criddle).

I = heat index (Thornthwaite).

L = evaporating power of the air (Turc).

L_A = daylight factor (Thornthwaite).

L_H = latent heat.

L_s = length of intake sample (cm).

m = transformation ratio $\sqrt{k_h}/\sqrt{kv}$

m_c = cloud cover factor.

μ = pore water pressure.

n = actual hours of sunshine.

n_g = number of wells in the group.

N = possible hours of sunshine.

PE = potential evapotranspiration.

P_H = penetration of well screen into aquifer.

$P_{m,a}$ = mean monthly, annual precipitation.

P_o = effective overburden pressure.

q = flow in casing.

Q_o = steady state flow in unit time.

Qw = discharge per well.

Qw_i = discharge from i^{th} well.

r = distance to observation well (m).

r_i = distance from i^{th} well to position at which drawdown is calculated.

r_s = reflection coefficient for earth's surface.

R, R_o = integration constants.

R_A = short wave radiation at outer limit of atmosphere.

R_c = short wave radiation actually received at earth's surface on a clear day.

R_g = radius of influence of well group.

R_i = radius of influence of i^{th} well.

RN = total of long and short wave radiation.

ρ = settlement.

ρ_f = density of fresh water.

ρ_s = density of sea water.

s = drawdown of water table (m).

S = surface run off.

$S_{A,G}$ = sensible heat to atmosphere, ground.

σ = total stress.

$\bar{\sigma}$ = effective stress.

δc = Limmer & Pringsheim constant

t = time interval in field permeability test.

T = basic time lag.

Ta = temperature ^{o}K.

T_A = mean annual temperature ^{o}C.

T_M = mean monthly temperature ^{o}C.

T_s = surface temperature ^{o}F.

u = fraction of Rc used in photosynthesis.

u_Z = wind speed at height Z.

u_z = wind speed at 2m.

v = volume required to equalize pressure difference in time lag equation.

y = distance from piezometer level to reference level in transient state (cm).

Z = height above sea level.

Z_i = depth below sea level to salt water/groundwater interface.

Z_o = height from piezometric level to reference level (cm).

The Regional Effect of Water Table Lowering in the Durban Area.

1.0 Introduction

The rapid growth of the City of Durban during recent years has resulted in the escalation of land costs, particularly in the central area. The viable development of building sites has therefore necessitated the construction of high rise buildings, but these in turn have resulted in an increased demand for parking facilities in the City centre. However, the provision of these facilities above ground is not always desirable, particularly if the lower floors are to be used as banking halls, supermarkets, etc., and it has therefore become increasingly necessary to be able to build below ground level and make use of basement areas.

The high level of the water table in the Durban area has, until recently, inhibited the construction of basements in excess of one level below ground. However, with the development and availability of advanced techniques, the construction problems are being overcome with the result that deeper basements are being constructed.

During construction it may be necessary to remove large volumes of water from the excavation in order to sufficiently lower the water table. Because of the density of building it is inevitable that the drawdown of the water table will affect adjacent properties; it may however have much more widespread and serious implications throughout the Durban area if carried out on a sufficiently large scale.

However, it is first necessary to determine whether such a lowering of the water table is possible and to investigate this the following points have been considered:

a. the existing groundwater levels,
b. the recharge to the catchment area,
c. the recharge to the City centre,
d. the permeability of the soil.

Once the possibility of drawdown has been established the effects of settlement and ingress of salt water from the nearby ocean due to reversal of flow are considered.

In view of the scope of the subject, the lack of any previous work on this subject in the Durban area and the lack of financial resources, this thesis has tended to a collection of existing data and a general approach to the problems rather than a detailed solution. Nevertheless, by taking into account the results of this work and the assumptions on which these have been based, an assessment may be made of the magnitude of the affects of any individual proposals for dewatering.

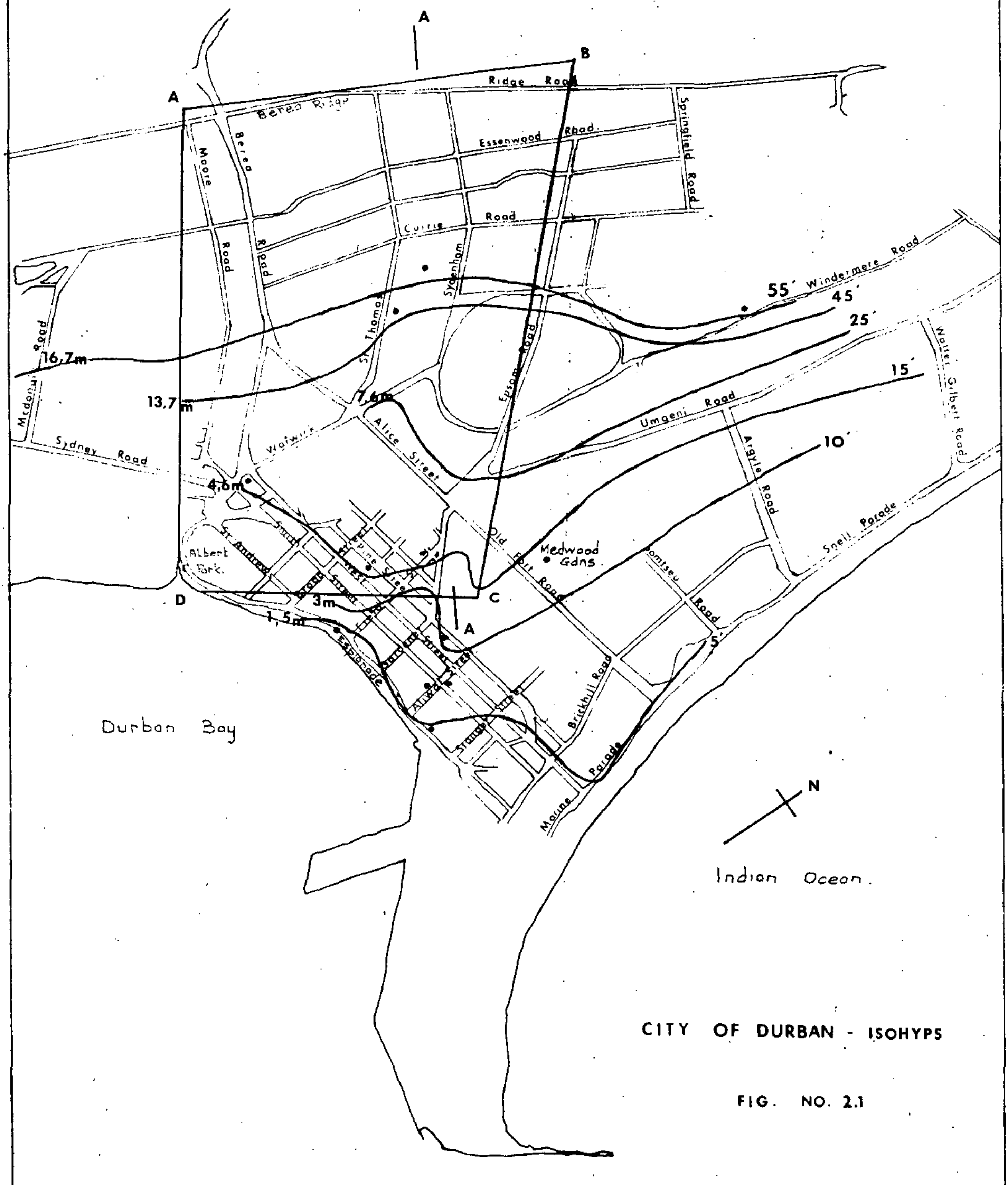
2.0 Existing groundwater levels.

Levels of the water table have been obtained by examination of borehole profiles and records taken in Durban by local contractors over a period of several years. The records from a large number of sites were studied and the water levels plotted onto large scale survey maps of Durban. Contours of the water level (isohyps) were drawn on these maps and transferred to 1":2500ft. maps. The results are given in Fig.2.1.

Local variations in water level due to differences in soil conditions, whilst being of importance on individual sites, were found to be insignificant to the overall situation.

Water levels recorded in boreholes carried out under the supervision of the author are shown by the heavy dots in Fig.2.1 and include the results from the experimental piezometers as described later in section 4.4. Apart from these positions, resources were not available for accurate observations and monitoring of existing water levels and the isohypal map is therefore not complete as most of the contractors' records related to the City centre, and extrapolation was necessary in the outer areas.

However the general trend can be quite clearly seen to be a flow south east from the Berea Ridge



CITY OF DURBAN - ISOHYPS

FIG. NO. 2.1

(see Fig.2.1). There was no information available on groundwater levels to the west of the Berea Ridge but on the assumption of equal infiltration (22) on both sides of the Ridge, the symmetrical shape of the Ridge and the semi infinite extent of the water carrying strata below the water table, it has been assumed that the subsurface water divide is equivalent to the surface water divide, i.e. a line which runs almost directly along Ridge and South Ridge Roads on the crest of the Berea Ridge. (See also Fig.3.1).

2.1 Variations in groundwater level.

2.1.1 Seasonal variations.

It was not possible to determine the extent of any seasonal variation from the contractors' previous records due to the short period over which the water levels were measured on most of the sites.

In the experience of the author, minor fluctuations (upto ± 100mm) do occur on some sites but this is dependent upon the local soil conditions.

2.1.2 Tidal variations.

In order to determine the influence of tides on the water table a series of readings were taken in a piezometer at position P5 in Fig.4.7 on 3rd April, 1973 and 17th May, 1973. These readings are given in Table 2.1 below. The times of high tide and the time to Spring or Neap tides are also given.

From Table 2.1 it can be seen that the maximum variation in level of the water table was recorded as 190mm. Although from a practical viewpoint this variation is not significant, it is interesting to note that the permeability of the soil is such as to almost eliminate the effect of tidal variations on the water table, even at positions very close to the sea. (P5 is approximately 40m from the shore).

The maximum and minimum levels of the water table were recorded from 2 to 5 days after Spring tide and from 2 to 5 days after Neap tides respectively.

The reason for the negligible effect of the tide on the water table may be due simply to the low permeability (especially in the vertical direction) of the natural deposits either in which the piezometer is located or which form the floor of the Bay itself. It is also considered probable that the original permeability has been lowered by the accumulation of oil and other waste material on the floor of the Durban

Bay and that this has further reduced the effect of tidal variations.

TABLE 2.1.

Influence of tide on water table.

Piezometer P5, Esplanade, Durban.

Date	Time	Depth below top of cover	High tide am	High tide pm	Remarks
20.3.73	1615	1,65	0431	1646	1 day after Spring tide
	1745	1,67			
21.3.73	1045	1,63	0457	1712	2 days after Spring tide
	1640	1,63			
23.3.73	1555	1,63	0545	1802	4 days after Spring tide 3 days before Neap tide
28.3.73	1215	1,71	1105	-	2 days after Neap tide
31.3.73	1245	1,73	0205	1417	5 days after Neap tide
3.4.73	0845	1,69	0336	1553	Spring tide
	1000	1,69			
	1120	1,68			
	1240	1,68			
	1435	1,68			
	1550	1,68			
4.4.73	1420	1,65	0408	1627	1 day after Spring tide
5.4.73	1120	1,58	0442	1702	2 days after Spring tide
	1730	1,54			
7.5.73	1130	1,55	0639	1908	5 days after Spring tide
17.5.73	0940	1,65	0336	1605	Spring tide
	1115	1,66			
	1245	1,66			
	1444	1,65			
	1610	1,65			

3.0 Groundwater Recharge.

In effect the problem is to calculate the water balance for the Durban area from the basic relationship:

Precipitation (P) - Evaporation & transpiration (E) - Run off (S) $\pm$ Groundwater gain or loss (G) $\pm$ Moisture content and storage (ΔM) = o

In order to do this for the area under consideration it is necessary to make certain assumptions due to the problems of determining the moisture content and storage capacity of the soil, and their effects on the potential evaporation and transpiration (i.e. the evaporation and transpiration when the moisture supply is unlimited). By choosing a time period for the water balance in such a way that the moisture content and storage are the same at the beginning and at the end of the period we can assume that ΔM=o. This could be expected to be the case for identical times in successive years and Van den Berg (12) is of the opinion that change in moisture content and storage may be ignored for a time period of one year. The equation then becomes

$$P - E - S \pm G = o$$

Variations in precipitation, percolation, temperature, wind and humidity all interrelate within this equation and it is not strictly permissible to calculate each term independently of the others and insert the values so obtained into the equation to find the unknown. However it is necessary to consider mean values if any attempt at a solution is to be made, and this has been done below.

3.1 Precipitation.

As stated above and also due to the need for most deep basements to have the water table lowered for long periods, the rainfall data used in the following is the mean monthly and mean annual measurement. The records which are given in Table 3.1 are from the rain gauge at the Botanical Gardens in Durban as supplied by the Department of Transport. The mean is calculated from a single gauge on measurements over the period 1871 - 1960 and the gauge has been unmoved over that period.

As an indication of the geographical position of the Botanical Gardens, piezometers P2 and P3 which are shown in Fig. 4.7 are located in the Botanical Gardens.

Irrespective of the other factors in the water balance equation it is obvious that periods of low rainfall, both seasonal and annual, will reduce the recharge to the area and subsequently increase the problems associated with drawdown of the water table. The effects of these variations in rainfall are extremely complex, linked as they are to the moisture content and evaporation & transpiration, and a complete solution is not yet

TABLE 3.1.

Mean Monthly Rainfall	
Month	Rainfall mm
January	113,7
February	121,9
March	127,4
April	79,6
May	50,5
June	32,5
July	27,2
August	39,1
September	72,0
October	106,9
November	119,8
December	122,6
	1013,2

possible even for well documented areas. However, an assessment of their importance may be made in relation to the mean annual effects which have been considered below.

3.2 Evaporation and Transpiration.

Evaporation is the conversion of water into water vapour and presuming that moisture is available two conditions must be met before it can occur.

1. There must be a supply of energy to provide the latent heat of vaporization. This is affected by:
   a. The percentage of cloud cover; evaporation generally being a maximum during daylight hours with no cloud cover.
   b. High temperatures; these have an effect on evaporation due to the increased energy which is available and also the air will absorb more water vapour as the temperature increases.
2. There must be a mechanism for removal of the vapour. This is affected by:
   a. Wind speed, which is a contributory factor in removing the saturated layer at the air/water interface thus permitting the inflow of dryer air.
   b. Humidity, which affects the ability of the air to absorb water vapour. High values of humidity in the surrounding air will reduce evaporation.

Transpiration is essentially the same as evaporation except that the surface from which the water molecules escape is not a free water surface. The water may be lost in a number of ways; by cuticular transpiration where water evaporates from moist membranes through the cuticle, by guttation where water is forced out through special organs called hydathodes, but by far the greatest water loss is by stomatal transpiration where the water escapes through numerous pores of stomata in the leaves. The dominant physiological factors are the density and behaviour of the stomata, the extent and character of the protective covering, the leaf structure and plant diseases. The physiological factors will not be discussed further here and reference should be made to the numerous works on plant physiology for further information.

The controlling environmental factors of transpiration are the same as for evaporation including temperature, solar radiation, wind and soil moisture content. In addition 95 percent of transpiration occurs during the hours of daylight (8).

Because of the practical difficulties of differentiating between evaporation and transpiration, both in estimates and in measurements in the field, the two processes are often considered together under the name of evapotranspiration.

When the moisture supply in the soil is limited plants have difficulty in extracting water and the actual evapotranspiration (E) falls below its maximum value, the potential evapotranspiration (P.E). The precise interdependence of the moisture supply and actual evapotranspiration is not known and the theories are controversial (24, 25, 37, 38, 39, 41). One view is that the potential rate is maintained until the soil-moisture content has dropped to some critical level below which the actual evapotranspiration decreases. Another view is that the evapotranspiration reduces as the soil moisture content reduces, the maximum potential rate being at maximum soil moisture content under conditions of free drainage.

Veihmeyer and Hendrickson (37-41) are of the opinion that the actual : potential evapotranspiration ratio is constant until plants are near the wilting point, whereas Thornthwaite & Mather (30) consider that the decrease below the potential rate is a logarithmic function of soil suction. Recent work by Chang (7) and Holmes (13) indicates that the ratio is approximately unity as long as the moisture content is not less than 75 percent of the maximum under conditions of free drainage. Work by Veihmeyer and Hendrickson (36, 40) also indicates that the significant loss of moisture when the soil moisture content is below maximum is due to transpiration and that when the soil surface is not in close contact (within 1m) with the water table, moisture movement is small and evaporation is negligible. This is also the conclusion of Ven Te Chow (9) and work by Call & Sewell (4) shows that the drying of the soil surface is beneficial to the prevention of loss of moisture by evaporation in that it reduces the points of contact between soil particles and hence lessens the capillary rise. Once this drying has occurred the evaporation loss, even over many months, is insignificant. In addition the use of mulching to retain moisture has been shown to increase the moisture loss by increasing the evaporation.

The soil type is also of importance and Chang (7) considers that Veihmeyer's and Hendrickson's results may apply to a heavy soil in a humid cloudy region but in sandy soils in an arid climate with vegetation cover a sharp decrease in actual evapotranspiration is possible.

In the Durban catchment area the surface deposits are almost entirely composed of a reddish brown silty sand known locally as Berea Red Sand. The top surface of this soil dries very quickly and in view of this a major water loss through evaporation alone would not be expected especially as the water table is well in excess of 1m below the ground surface.

In order to estimate the evapotranspiration in the area under consideration, estimates based on theoretical methods have been calculated, and these have then been compared to the values obtained by measurement from open pans.

None of the theoretical methods available have been tried and tested in the Durban area and because of this their applicability is in doubt for the estimation of absolute values. However, in the absence of local experimental results they are useful as a method of obtaining a rough estimate of the evapotranspiration values to be used.

3.2.1 Theoretical Methods.

Numerous methods have been developed for calculating the loss of moisture by evapotranspiration (1, 9, 17, 22-26, 33, 34, 40, 43) and some of the most well known are described briefly below.

a. Aerodynamic Method.

Penman (23) considered the removal of vapour from the evaporating surface. The controlling factors are the vertical gradient of humidity and the turbulence of the air flow. Evaporation from large water bodies is related to mean wind speed (u_z) at height Z, and to mean vapour pressure difference (e_s-e_z) between the water surface and the air at level Z. The evaporation from the open water surface E_o is then converted to potential evapotranspiration PE by the use of a conversion factor developed by Penman.

$$E_o = f\ (u_z)(e_s - e_z)$$

Further work has extended this equation to give

$$E_o = 0{,}033(e_s - e_z)\ u_2^{0{,}76}\ \text{mm/day}$$

Following a series of experiments at Rothamsted Experimental Station, Penman concluded that the best practical form of this equation is

$$E_o = 0{,}35(1+9{,}8\text{x}10^{-3}u_2)(e_s-e_z) \text{ mm/day} \qquad (1)$$

Applying the meteorological data in Table 3.3 to equation (1) gives the values for E_o shown in column 2 in Table 3.6.

b. Energy Budget Method.

This method was originally derived by Schmidt W. (1915) to obtain an estimate of evaporation from the oceans. It was later developed by others to be applied to lakes and other open water surfaces. From fundamental principles of the conservation of energy the net total of long and short wave radiation RN received at the earth's surface is available for three processes; the transfer of sensible heat (S_A) and latent heat (L_HE) to the atmosphere, and of sensible heat (S_G) into the ground.

$$RN = S_A + L_HE + S_G$$

The amount of RN used in plant photosynthesis may be considered negligible and evaporation may then be determined by measurement of the other terms

$$E = \frac{RN - S_A - S_G}{L_H}$$

RN may be measured using a net radiometer; S_G may be calculated from data on the soil-temperature profile or direct measurement of soil heat flux, and S_A may be estimated indirectly from Bowen's ratio $B_o = \frac{S_A}{L_H E}$. This is calculated from the ratio of the vertical gradients of temperature and vapour pressure.

Substituting B_o into the above equation gives

$$E = \frac{RN - S_G}{L_H(1+B_o)}$$

c. Temperature Methods.

Thornthwaite (30) estimates potential evapotranspiration (PE) by relating observations of consumptive use of water in irrigated areas to air temperatures with adjustments for day and month length.

$$PE\ \text{mm/month} = 16(L_A)\left(\frac{10\ T_M}{I}\right)^a$$

where T_M = mean monthly temp ^{o}C

a = an empirical function of I

I = a heat index, which is the sum of 12 monthly indices j,

where $j = \left(\frac{T_M}{5}\right)^{1,514}$

L_A = an adjustment for number of daylight hours and month length.

Checking by field trials is essential and Thornthwaite himself is of the opinion that the formula is not valid for climates other than that for which it was developed.

Turc's method (33, 34) relates air temperature and rainfall to the annual evapotranspiration (E) in an attempt to allow for the limiting effect of water supply.

$$E = \frac{P_A}{\left[0,9 + \left(\frac{R_A}{L}\right)^2\right]^{\frac{1}{2}}} \text{ mm/annum}$$

where P_A= annual precipitation mm

$L = 300 + 25T_A + 0,05\ T_A{}^3$

T_A= mean annual air temperature $^{\circ}C$

A second more complicated formula has been developed by Turc to take into account the soil moisture supply over short periods (10 days) but neither method has been tested for reliability. As an exercise, however, the precipitation (1013mm) and temperature (20,3°C) for Durban have been substituted into the above formula which indicates an actual evapotranspiration of 810mm.

d. Empirical Method.

Blaney & Criddle (1) have developed a purely empirical approach to relate evaporation to temperature, relative humidity and length of daylight.

$$PE = 25,4\ K_B F_C \text{ mm}$$

where K_B = empirical constant depending on type of crop

F_c = the sum of monthly consumptive use factors i.e. product of mean monthly temperature (^oF) and monthly percentage of day time hours of the year

The potential evapotranspiration in one area, with a particular crop, is measured and this is used to estimate the evapotranspiration, with the same crop, in some other area. The results are not consistent and the method can only be used as a rough guide. For accurate results K_B must be known for the time and place for each area where this method is applied.

e. Combination Method.

A number of methods which combine the aerodynamic and energy budget approaches have been developed and perhaps the most widely used and tested is that by Penman (23, 26) which is described below. This method is later used to give an estimate of the potential evapotranspiration.

As stated earlier the relationship between Potential Evapotranspiration, available moisture, and actual evapotranspiration is controversial. In terms of experimental work the conclusions of Veihmeyer and Hendrickson described above appear to be the most satisfactory. They consider that the rate of moisture extraction is not influenced by the amount of moisture in the soil when the soil moisture is above the wilting percentage. Whilst this may be the case for transpiration it does not hold true for evaporation from the soil surface and accordingly a factor to allow for this is applied to the potential evapotranspiration estimate given by Penman.

Penman's method assumes that if continuous evaporation is to occur, there must be a supply of energy to provide latent heat of vapourization and there must be a mechanism for the removal of the water vapour produced.

During daylight hours a certain measurable amount of solar radiation arrives at the earth's surface. The amount depends on latitude, season, time of day and cloud cover. Angot (3) has given values of short wave radiation flux R_A at the outer limit (8 - 16Km) of the atmosphere as a function of the month of the year and the latitude. Values of R_A for Durban are given in Table 3.2.

R_A for Durban in gramme calories per cm² per day

TABLE 3.2.

Month	R_A
January	984
February	990
March	760
April	625
May	484
June	444
July	460
August	565
September	734
October	850
November	990
December	1005

Using as the unit of energy the amount required to evaporate, 0,1g water at air temperature, it is possible to arrive at the following expression for the heat budget H taking account of the incoming short wave radiation from sun and sky and the long wave exchanges between earth and sky.

$$H = Rc\ (1-r_s-u)\ -\ \sigma_c T_a^4\ (0{,}56-0{,}092\ \sqrt{e_d})\ (1-0{,}09m_c) \qquad (2)$$

where

Rc = short wave radiation actually received at earth's surface on a clear day /cm^2/day

r_s = reflection coefficient for the surface

u = fraction of Rc used in photosynthesis
= 0,005 and is therefore negligible

$\sigma_c T_a^4$ = theoretical black body radiation at T_a in ^{o}K

ed = saturation vapour pressure *mm Hg* at dewpoint

$m_c/10$ = fraction of cloud cover

The heat budget H can be shown to be

$$H = E_o + K \text{ to a good approximation} \tag{3}$$

where

K = convective heat transfer from the surface.

From an empirical approach

$$E_o = (es - ed)\ f(u)$$

where

E_o = evaporation in unit time

es = saturation vapour pressure at evaporation surface temperature

f(u) = function of horizontal wind velocity.

Penman (23, 26) assumed the transport of vapour and the transport of heat by eddy diffusion were essentially controlled by the same mechanism. One is governed by (es - ed) and the other by (Ts - Ta).

where Ts, Ta = surface and air temperature respectively ^{o}F.

Therefore to a good approximation

$$K/E_o = \beta = \gamma (Ts - Ta)/(e_s - ed) \qquad (4)$$

where

$$\gamma = 0,27 \text{ in } ^{o}F \text{ and mm Hg}$$

now

$$H = E_o + K = E_o (1 + \beta) \qquad (4A)$$

therefore

$$E_o = \frac{H}{1 + \beta} = \frac{H}{1 + \gamma (Ts - Ta)/(e_s - ed)}$$

The Rc term in equation (2) can be estimated for periods greater than one month from duration of sunshine and Angot has given tables (see Table 3.2) of total radiation R_A to be expected if the atmosphere were perfectly transparent (3). There appears to be a general correlation between Rc/R_A and n/N the variation of actual/possible hours of sunshine. This is of the form

$$Rc/R_A = w + xn/N$$

Angstrom (3) has estimated this to be a maximum of

$$Rc = R_A (0,25 + 0,75 \, n/N)$$

However, of this total amount approximately 6 - 8 percent is absorbed in the atmosphere, 9 percent is reflected back from the atmosphere and 8 percent reflected back from the earth surface.

Assuming that for Durban w = 0,25 and x = 0,54 then

$$Rc = R_A \left(0{,}25 + 0{,}54 \frac{n}{N}\right) \tag{5}$$

r_s will vary with type of surface and season; for a freewater surface its annual mean will be approximately 0,06 and for turf approximately 0,2.

The major uncertainty that exists is in the estimation of the cloudiness fraction since cloud control of solar radiation must depend upon cloud type. To make some allowance for this it is proposed to set

$$\frac{m_c}{10} = 1 - \frac{n}{N}$$

equation (2) then becomes

$$H = E_o(1 + \beta) = (1-r_s)(R_A)\left(0{,}25 + 0{,}54\frac{n}{N}\right) - \sigma T_a^4 \left(0{,}56 - 0{,}092 \sqrt{ed}\right)\left(0{,}10 + 0{,}90\frac{n}{N}\right) \tag{6}$$

The parameters on the RHS of equation (6) are all readily determinable but to obtain E_o it is necessary to obtain β which requires a knowledge of the surface temperature (equation 4). This can be eliminated as follows:-

$$E_o = (es - ed)\, f(u)$$

if Ea replaces E_o by replacing es by ea then

$$Ea = (ea - ed)\, f(u)$$

where

ea = saturation vapour pressure at air temperature i.e. $\dfrac{Ea}{E_o} = 1 - (es - ea)/(es - ed) = 1 - \emptyset$ say (7)

from (4) and (4A)

$$E_o = H/(1 + \beta)$$

$$= H/(1 + \gamma(Ts - Ta)/(es - ed))$$

if $Ts - Ta = (es - ed)/\Delta$

where

$$\Delta = \text{slope of } e : T \text{ curve at } T = Ta$$

then

$$H/E_o = 1 + \gamma(es - ea)/\Delta(es - ed)$$

$$= 1 + \gamma\emptyset/\Delta \quad (8)$$

from (7) and (8)

$$E_o = (H\Delta + Ea\gamma)/(\Delta + \gamma) \quad (9)$$

$$es = (ea - \emptyset ed)/(1 - \emptyset)$$

E_o may now be calculated from information available from standard meteorological observations of mean air temperature, mean dew points, mean wind speed at a standard height and mean duration of sunshine. For Durban these data are set out in Table 3.3 below:-

TABLE 3.3.

Meteorological data (Durban)

Month	°K	Relative Humidity $h=\frac{ed}{ea}$ %	Sat.vap. pressure es (=ea)	Mean wind speed in mph	B	Mean daily sunshine hours	n/N
Jan.	296,8	81	22,1	7,7	3	6,5	0,42
Feb.	297	81	22,27	6,8	3	6,7	0,46
Mar.	296,5	81	21,71	6,6	3	6,2	0,46
April	294,1	79	18,77	5,5	2	6,8	0,55
May	291,5	76	15,96	5,2	2	7,4	0,65
June	288,9	72	15,44	4,9	2	6,9	0,65
July	289,1	73	13,71	5,3	2	6,9	0,63
Aug.	290,3	76	14,8	7,3	3	6,9	0,58
Sept.	291,8	78	16,26	8,5	3	5,8	0,49
Oct.	292,9	79	17,43	8,6	3	5,2	0,37
Nov.	294,3	81	19,0	9,2	3	5,7	0,40
Dec.	295,9	81	20,93	8,1	3	6,0	0,39

where B = Beaufort wind force.

In equation (6) we can assume that $r_s = 0{,}06$

therefore $1 - r_s = 0{,}94$

Values of Ra are given in Table 3.2

and σ_c the Limmer and Pringsheim constant is taken as

$\sigma_c = 117{,}74 \times 10^{-9}$ g cal./cm^2/day.

Therefore from equation (6) we can calculate the following values for the heat budget H.

TABLE 3.4.

Monthly Values of Heat Budget H

Month	H
January	366,47
February	384,13
March	274,55
April	197,27
May	133,72
June	97,65
July	107,70
August	168,38
September	247,94
October	277,80
November	352,83
December	360,25

From which assuming $\Delta \doteqdot 1{,}07$ at 20°C and according to Penman (23)

$Ea = 0{,}37B\ (ea - ed)$ mm/day to a good approximation where B = Beaufort wind force.

Values of Ea are given in Table 3.5 and values of E_o from equation (9) are given in column 3 in Table 3.6.

3.2.2 Local records of evaporation.

Local records of evaporation are kept at Louis Botha Airport and at the Mount Edgecombe Sugar Research Association.

The Mount Edgecombe records are from the Symons pan and the mean has been calculated over a 39 year period during which time the pan has not been moved.

The Louis Botha Airport records are also for the Symons pan and the mean monthly gross evaporation has been calculated over the period March 1957 to August 1971.

The records for both stations are given in Table 3.6.

TABLE 3.5.

Values of E_a mm/day

Month	E_a mm/day
January	4,66
February	4,71
March	4,58
April	2,92
May	2,84
June	2,81
July	2,74
August	3,94
September	3,97
October	4,06
November	4,01
December	4,41

3.2.3 Evapotranspiration - assumed values.

A summary of the theoretical and recorded values of evaporation E_o is given in Table 3.6.

The theoretical values obtained from the aerodynamic approach are generally closer to the recorded values than those obtained by the Penman combined approach and the latter are considerably lower throughout the year.

A comparison between the total yearly evaporation from the aerodynamic approach and the Mount Edgecombe records shows that the aerodynamic values are higher by only 6,4%, the theoretical values being generally higher in the winter and lower in the summer. At this stage, however, any agreement in the values should be considered to be coincidental.

Louis Botha records consistently higher values of evaporation than Mount Edgecombe. The reason for this is considered to be the more exposed position of the airport. Topographically, the study area on the slopes of the Berea and downtown Durban is rather more similar to Mount Edgecombe than to the airport and because of this, the Mount Edgecombe recorded values have been assumed to apply to Durban.

It is accepted that for this work these values may be as misleading as the others, but it should be recalled that the intention is to determine the order of magnitude of the problem and that even in well documented areas the prediction of absolute values of evapotranspiration is not yet possible.

From Penman (23, 26) the values of ET/E_o given in Table 3.7 may be assumed for Durban and f_D is a correction factor for local con-

TABLE 3.6.

Values of E_o : theoretical, recorded and assumed values

Month	E_o theoretical mm		E_o recorded mm		E_o
	Aero-dynamic Method	Combined Method	Mount Edgecombe S.R.A	Louis Botha Airport	Assumed Value mm
Jan.	128	96	148	161,9	148
Feb.	108	112	126,9	144,6	127
March	115	96	120,9	140,6	121
April	96	66	90,4	105,7	91
May	93	50	72,6	85,0	73
June	97	42	61,7	67,1	62
July	91	43	64,6	68,2	65
August	105	65	77,2	90,8	78
Sept.	113	84	94,1	103,6	94
Oct.	120	93	109,5	124,6	110
Nov.	120	108	123,0	127,6	123
Dec.	125	118	142,3	163,6	142

TABLE 3.7.

Evapotranspiration factors

Month	ET/E_o	f_D	ET/E_o revised
January	0,8	0,8	0,64
February	0,8	0,8	0,64
March	0,8	0,8	0,64
April	0,7	0,7	0,49
May	0,7	0,7	0,49
June	0,6	0,6	0,36
July	0,6	0,6	0,36
August	0,6	0,6	0,36
September	0,7	0,7	0,49
October	0,7	0,8	0,56
November	0,8	0,8	0,64
December	0,8	0,8	0,64

TABLE 3.8.

Evapotranspiration loss and rainfall

Month	Evapotranspiration loss E mm	Rainfall mm
January	95	113,7
February	81	121,9
March	77	127,4
April	44	79,6
May	36	50,5
June	22	32,5
July	23	27,2
August	28	39,1
September	46	72,0
October	61	106,9
November	79	119,8
December	91	122,6
	683 mm/yr.	1013,2 mm/yr.

ditions and the fact that there is not always a plentiful supply of water, an assumption on which the test and observed results depended.

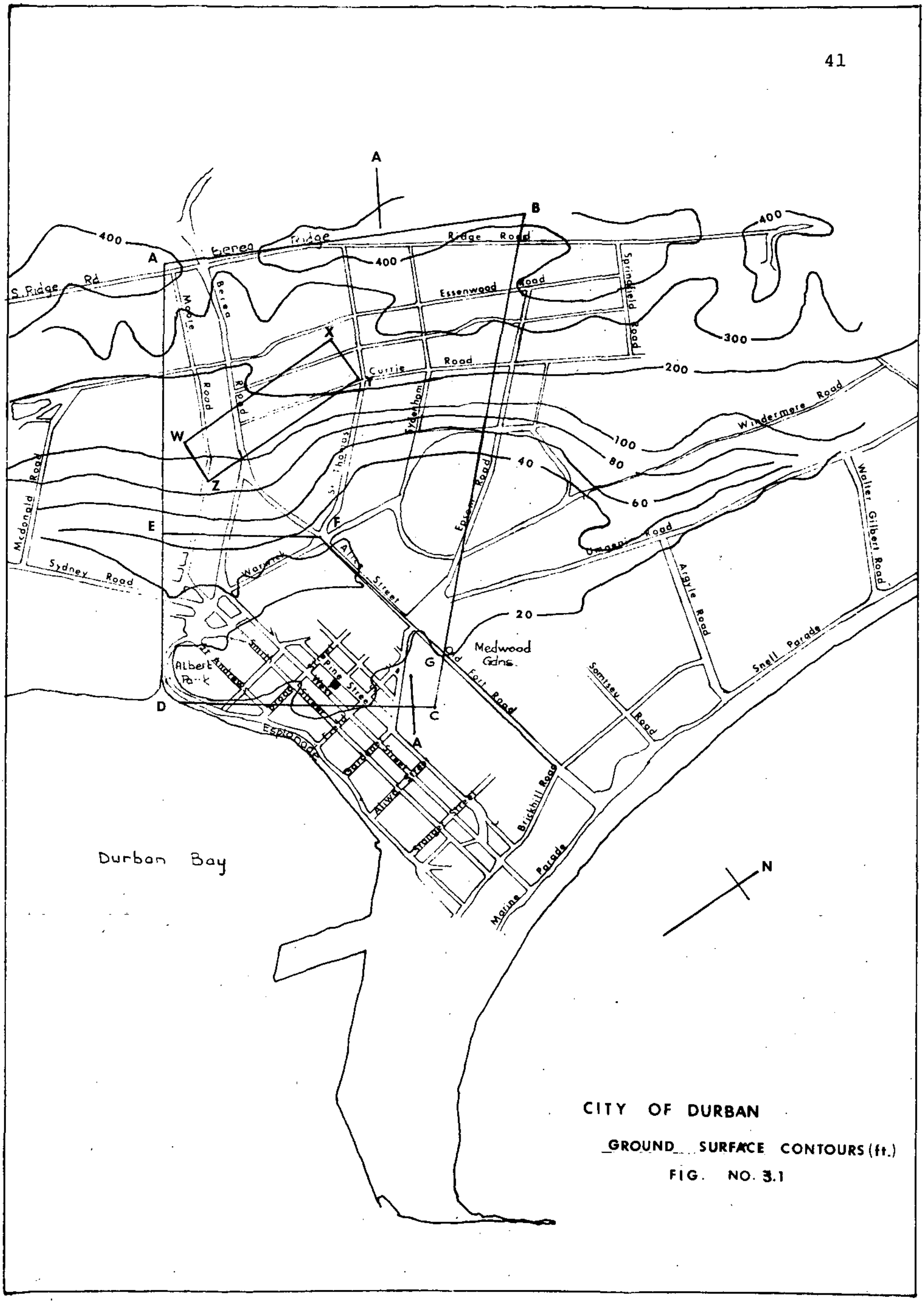
Using the revised ET/E_o values in Table 3.7 ($\frac{ET}{E_o}$ revised = $\frac{ET}{E_o} \times f_D$) the values of E have been calculated from $E = \frac{ET}{E_o}$ revised $\times E_o$. The results are given in Table 3.8 and the total evapotranspiration loss is 683 mm/year. As a comparison the annual evapotranspiration predicted by Turc's formula was 810 mm/year.

Assuming that the annual evapotranspiration is 683 mm/year and subtracting this from the annual precipitation, then the infiltration is 330 mm/year.

3.3 Surface run off.

On the assumption that groundwater flow is normal to the isohyps an estimate may be made for the size of the catchment area for flow across a line CD which extends through the centre of the City (Fig.3.1). This assumption presupposes that the percolation to the water table is also normal to the isohyps and that no recharge or loss occurs across the boundaries.

The subsurface water divide on the Berea Ridge is taken to be line AB and this determines the NW extent of the catchment. The other boundaries AD and BC are drawn normal to the isohyps in Fig.2.1 to



CITY OF DURBAN

GROUND SURFACE CONTOURS (ft.)

FIG. NO. 3.1

intersect the line CD as shown. The catchment area assumed is then ABCD in Figs.2.1 and 3.1.

The area CDEFG in Fig.3.1 is almost entirely paved and built up and is therefore assumed to have a 95 percent surface run off factor (9).

In order to determine the losses due to surface run off on the remaining area ABGFE, a dot count was carried out on a 200ft. to one inch map of the Durban area between grid references +840000 and +836000, and ± 0 and -1000. This area is shown approximately on Fig.3.1 as WXYZ and whilst it is not possible to prove by calculation that this is a typical area within ABGFE a careful study of large scale maps of the area coupled with personal knowledge confirm that the area is representative of the area generally. Basically it consists of a mix of flats and houses with a cross section of house and garden sizes. The area is reasonably covered by roads and footpaths together with parks and grassed and domestically cultivated areas. A few public buildings also occur.

All paved areas and all areas covered by buildings are drained to Durban Corporation stormwater drainage and are considered to have a surface run off factor of 95 percent (9).

Assuming then that the area WXYZ is typical of the catchment area ABGFE, the dot count indicates that 40,6 percent of the catchment area is covered by paving or buildings.

The area available for infiltration is therefore 61,43 percent of area ABGFE in Fig.3.1, and according to Midgley (22) this area is subject to 100 percent infiltration.

3.4 Recharge.

For flow across CD in Fig.3.1 the catchment area is 61,4% area ABGFE + 5% area CDEFG

$= 0{,}614 \times 4{,}7 \times 10^6 + 0{,}05 \times 1{,}85 \times 10^6$ m²

$= 2{,}89 \times 10^6 + 0{,}093 \times 10^6$ m²

$\doteqdot 2{,}98 \times 10^6$ m²

The infiltration is 330 mm/year (Table 3.8) resulting in a total recharge across CD of

$2{,}98 \times 10^6 \times 0{,}33$ m³/year

$= 0{,}983 \times 10^6$ m³/year

$= 2{,}7 \times 10^3$ m³/day.

4.0 Permeability

The extent to which the water table may be lowered is dependent upon the permeability of the soil. Since it is not realistic, because of the large local variation in soil profile (see BH nos. 1-8 in Appendix A), to calculate a permeability for a localised area or soil strata and apply it generally to any site in Durban, an estimate for an average permeability for the Durban area must be made in order to determine the larger scale effects of dewatering.

This value may be determined by one, or a combination, of the following approaches:-

1. From soil profiles, isohyps and recharge.
2. Test pumping in the field.
3. Laboratory tests on disturbed and undisturbed samples.
4. Permeability tests in the field.

4.1 Permeability from soil profiles, isohyps and recharge.

The Darcy formula $Q_o = K_a H_s \frac{dH_s}{dx}$ is applied to give an indication of the order of magnitude of the coefficient of permeability. Provided certain assumptions on the conditions of flow and the soil profiles are made, each of the terms in the Darcy equation may be calculated using the information previously derived on the isohyps and recharge.

The following assumptions have been made for flow across CD along AA in Fig.3.1:-

a. steady flow,

b. flow of phreatic water through an unconfined aquifer above an impervious base.

From a study of the flow lines, the flow is considered to be in a direction normal to line CD which passes through the area under investigation and the analysis is therefore reduced to that of a two dimensional problem.

Further, the soil profile is essentially that of a layered system; the clayey and silty layers of which have very low permeabilities. It has been shown (28) that under these conditions the governing flow occurs through the permeable sandy layers and vertical flow is negligible.

Under conditions of recharge, the flow lines are no longer parallel to each other; however, provided that horizontal dimensions are large compared to vertical dimensions, the divergent angle between two adjacent flow lines is small and the component of flow perpendicular to the main direction of movement remains negligible.

If the thickness of a repeated layer is small in comparison to the length of the drainage path (infite layer system) the individual layers can be treated as an homogeneous and anisotropic medium, the permeability of which is taken as equal to the average values for the repeated soil layers.

A good approximation of actual conditions may therefore be obtained by assuming that the flow is one dimensional, and is governed by an average permeability k_a.

Assuming then that the flow along a line AA in Fig.3.1 is unidirectional, and considering unit width, the total recharge G to this strip before it reaches line CD in Fig.3.1 is

$$G = (\text{infiltration}) \times (\text{total catchment area})$$

$$= 0,330 \times 2590 \times 0,614 + 0,05 \times 610 \times 0,330$$

$$= 535 \text{ m}^3/\text{year}$$

$$= 1,69 \times 10^{-5} \text{ m}^3/\text{sec}$$

assuming steady flow

$$G = Q_o = k_a \frac{H_s dH_s}{dx}$$

where

k_a = coefficient of permeability

H_s = saturated thickness

$\frac{dH_s}{dx}$ = hydraulic gradient

and according to Fig.2.1

$$\frac{dH_s}{dx} = \frac{1,52}{572} = ,00266$$

at CD according to borehole date

$$H_s \doteqdot 26\text{m}$$

therefore

$$k_a = \frac{1,69 \times 10^{-5} \times 100}{2,66 \times 10^{-3} \times 26} = 2,44 \times 10^{-2} \text{ cm/sec}$$

Despite the number of assumptions on which the above determination of k_a is based, this method can be used to arrive at an order of magnitude. Using the limits of infiltration to determine the probable limits of permeability, then assuming only 1mm of rainfall available for recharge

$$k_a = 7,39 \times 10^{-5} \text{ cm/sec}$$

and at the other extreme asuming no loss to evapotranspiration

$$k_a = 7,49 \times 10^{-2} \text{ cm/sec}$$

The other variables are defined between reasonably close limits and would not significantly affect the limits of permeability given above.

One aspect which may affect the above however is the former course of the Umgeni River. According to King and Maud (18) the Umgeni River at one time discharged into the Natal Bay at about the position of Albert Park (see Fig.3.1), and it is possible that this course still affects the recharge to this area. The material which was deposited in the Umgeni River bed will certainly be different to the alluvial deposits beneath the remainder of Durban but it is not known if there is any significant difference between the permeabilities of the two materials. If the river deposit is the more permeable, then the former river course may act as an underground drain cutting off flow to the east and discharging beneath the City or into the bay.

If the converse is true and the deposits in the former course of the river are less permeable a restriction to the flow to the east could also occur.

Unfortunately finance was not available for any field work and therefore proof of the existence and affect of this point was not possible.

However, a study of the existing water levels (Fig.2.1) does not show any sign of specific irregularities in the area of the former river and its affect may therefore be of little significance.

4.2 Permeability from field pumping tests.

During November and December, 1970, a pumping test was undertaken in the centre of Durban in order to determine the actual drawdown and quantity of flow in the field. The site is located between West Street and Pine Street, and Field Street and Broad Street, and is shown by the solid rectangle in Fig.3.1.

The test well was a 6" (150mm) I.D. borehole pumped by a vacuum pump at various depths in order to assess the variation of permeability with depth. The flow was measured by recording the time required to fill a 40 gallon (182 litre) drum.

Standpipes with filters at depths of 25ft. (7,6m) and 50ft. (15,2m) were located in two directions at right angles as shown on the site plan Fig.4.1. The soil profile adjacent to the test well is given by BH.1 (in Appendix A), the position of which is also shown on the site plan Fig.4.1. BH.2 and BH.3 were located approximately 300ft. (90m) from the test well. Readings of the water level in the standpipes were measured with

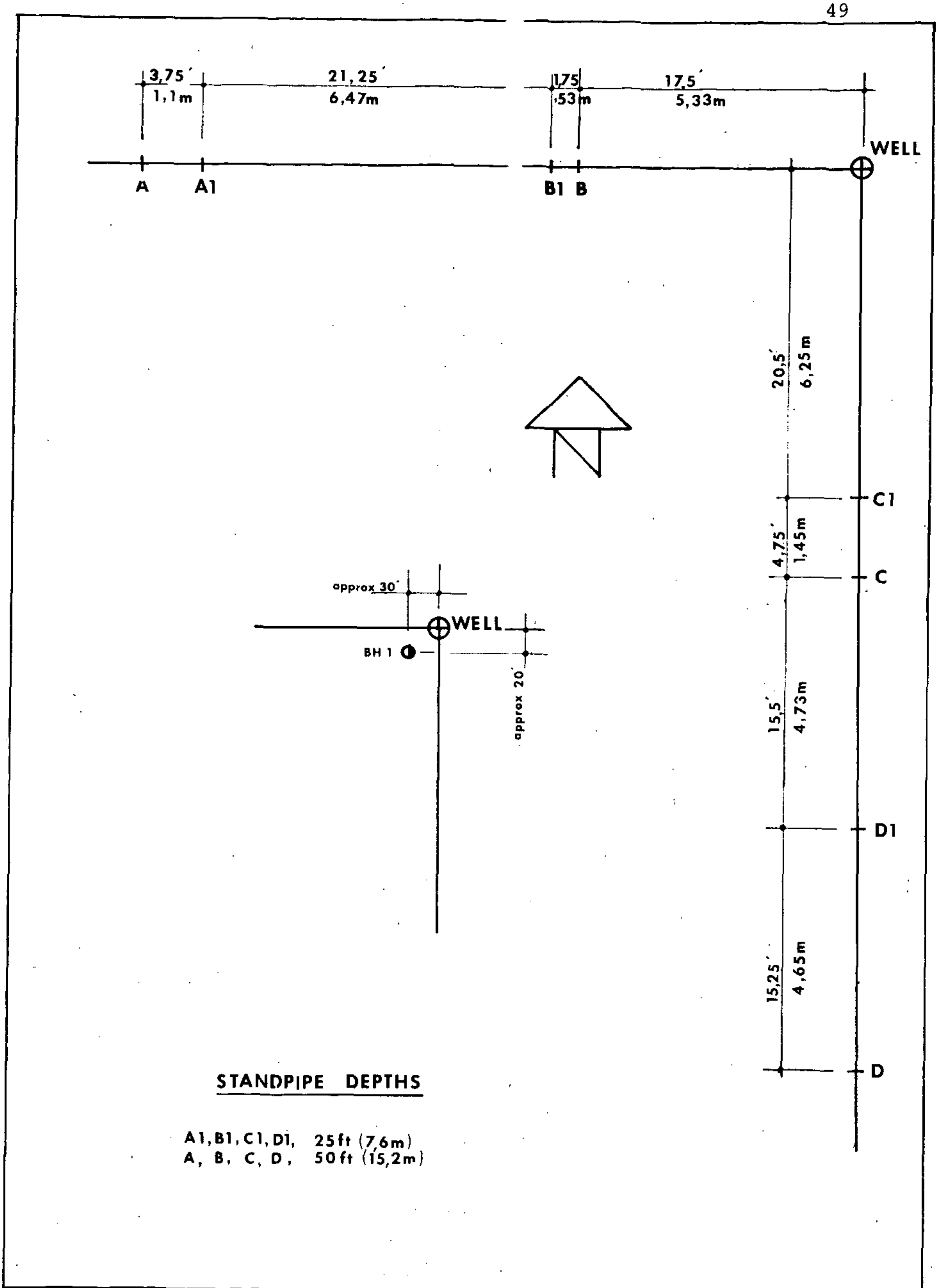


FIG. 4.1
SITE PLAN : POSITION OF WELL & STANDPIPES & BH 1.

an electrical dipper during pumping and recovery, and these, together with the flow and the times when the readings were taken, are given in Tables 4.1 - 4.3.

The well was pumped from two different levels at different times: viz. from 28ft. (8,5m) and 50ft. (15,2m) below ground level.

4.2.1 Pumping at 28ft. (8,5m).

From Table 4.1 it can be seen that the flow from the well at this level was very low and it was only possible to pump intermittently during the test.

The drawdown shows considerable variation across the site and is generally quite small for standpipes at both levels. However, at standpipe B1 a lowering of approximately 5ft. (1,5m) is recorded, but the effect at B which is adjacent to B1 is much less. This high local variation is difficult to explain but it is probably due to the horizontal and vertical variations in permeability across the site. The reason that a similar variation in drawdown was not observed in standpipe C could also be that this piezometer was located in clayey material which may have encroached to the 25ft. (7,6m) level at this position. It is also possible that a leaky confined aquifer exists at approximately 50ft. (15,2m) caused primarily by the clay layer which is shown in BH.1 between 38ft. (11,6m) and 43 ft. (13,1m).

PUMPING TEST RESULTS
(Depth to water table ft.-in.)

PUMPING DEPTH 28'-0" (8,5m) TABLE 4.1.

DATE	10.12.70				11.12.70		14.12.70			15.12.70					
TIME / STANDPIPE	8.30	10.50	2.00	4.00	8.30	3.30	8.30	12.30	3.30	8.45	12.30				
A	5-11	5-11	5-10	5-10	5-10	6-8	6-10	7-1	7-1	7-1	7-1				
A1	4-8	4-8	4-8	4-11	4-9	4-9	4-10	5-0	5-0	5-2	5-2				
B1	5-7	6-2	6-2	6-11	6-11	7-6	6-8	8-5	8-5	10-7	10-6				
B	9-1	9-1	9-3	9-6	9-11	9-11	9-2	9-4	9-6	10-2	10-2				
C1	5-2	5-5	5-6	5-8	5-8	5-11	5-7	6-1	6-2	7-1	7-1				
C	9-2	9-3	9-7	9-10	10-0	10-1	9-5	9-10	9-12	10-6	10-8				
D1	5-0	5-0	5-1	5-2	5-1	5-1	5-1	5-2	5-2	5-4	5-4				
D	9-8	9-8	9-9	9-11	10-0	10-1	9-9	9-10	9-11	10-5	10-5				
RATE OF FLOW Gal / sec		.080	.088	.127	.029	.028	.015	.047	.059	.059	.055				
REMARKS	Machine switched on at 10 am. on 10.12.70. Intermittent pumping thereafter until 12.50 on 15.12.70.											Machine switched off at 12.50 pm.			

The existence of a confined aquifer is also indicated by the difference in original water levels.

The choice of analysis of the results is complicated by the variation in conditions across the site and also by the suggestion of a leaky confined aquifer. If the analysis proceeds on the assumption of a leaky confined aquifer the problems are to assess the thickness and the value of the resistance to vertical flow of the semi permeable layer separating the two aquifers. Because of the difficulty in assessing these constants the analysis is based on the assumption of pumping from a fully penetrating well in a confined aquifer which is bounded at 18ft. (5,5m) and 28ft. (8,5m) by impervious layers (see BH.1).

In general the flow of water to a well is three dimensional; however, in the case of a confined aquifer the flow to a fully penetrating well may be assumed to take place in horizontal planes and the vertical component of flow assumed to be zero. In the immediate vicinity of the well a radial flow is present which in polar co-ordinates is unidirectional. Thus the properties of flow, velocity, piezometric head, etc. all

PUMPING TEST RESULTS
(Depth to water table ft.-in.)

PUMPING DEPTH 50'-0" (15,2m) TABLE 4.2.

DATE	23.11.70					24.11.70			25.11.70			26.11.70		27.11.70	
TIME / STANDPIPE	11.00	1.30	2.00	3.30	7.00	8.00	12.00	3.30	8.30	12.30	3.30	8.30	3.30	8.30	3.30
A	8-6	8-9	8-10	9-0	9-4	10-7	10-11	11-2	10-11	10-9	10-4	8-11	8-7	8-5	8-3
A1	4-9	4-11	5-0	5-4	5-8	6-7	6-8	7-0	6-10	6-7	5-5	5-2	5-0	5-0	5-0
B1	5-8	6-3	6-10	7-3	7-8	8-8	9-2	9-6	9-0	8-4	7-8	6-7	6-5	6-2	6-0
B	8-11	10-8	11-0	11-10	14-0	19-0	19-11	20-7	19-0	16-6	14-8	10-8	9-11	9-4	9-2
C1	5-8	6-1	6-2	6-4	6-0	6-4	6-5	6-5	6-0	5-10	5-8	5-8	5-8	5-6	5-5
C	9-0	11-10	12-2	13-1	15-6	20-5	21-1	21-7	18-9	15-5	13-4	10-2	9-9	9-3	9-2
D1	4-10	4-10	5-4	5-4	5-4	5-4	5-4	5-4	5-2	5-2	5-2	5-2	5-2	5-2	5-2
D	9-9	10-8	10-10	11-5	12-8	16-4	16-11	17-4	16-10	14-8	13-3	10-7	10-2	9-11	9-10
RATE OF FLOW Gal / sec	.942	.667	.887	.642	.618	.556	.550	.538							
REMARKS			10 min. breakdown						5.30 am. Machine out of order during this period - see Table 4.3.						

PUMPING TEST RESULTS
(Depth to water table ft.-in.)

PUMPING DEPTH 50'-0" (15,2m) TABLE 4.3.

DATE	30.11.70			1.12.70			2.12.70			3.12.70		4.12.70		7.12.70	
TIME / STANDPIPE	8.30	12.30	3.30	8.30	12.30	3.30	8.30	12.30	3.30	8.30	12.30	11.30	3.30	10.30	3.30
A	12-1	12-3	12-4	12-11	12-11	12-11	12-9	12-9	12-9	12-9	12-9	8-1	7-9	7-1	7-1
A1	6-0	6-3	6-5	6-8	6-9	6-10	6-7	6-7	6-7	6-9	6-9	5-1	5-0	4-8	4-8
B1	7-10	9-0	9-5	9-7	9-9	9-9	9-5	9-6	9-6	9-7	9-7	7-2	6-9	5-9	5-9
B	22-0	22-1	22-1	22-3	22-3	22-3	22-5	22-5	22-5	22-6	22-6	11-0	10-5	9-0	9-0
C1	6-0	6-2	6-3	6-3	6-3	6-3	6-0	6-0	6-0	6-1	6-2	5-10	5-4	5-2	5-2
C	20-9	20-11	21-0	21-2	21-2	21-2	21-2	21-2	21-2	21-2	21-2	10-4	10-0	9-1	9-1
D1	5-3	5-3	5-3	5-3	5-4	5-4	5-4	5-4	5-4	5-4	5-4	5-3	5-2	5-1	5-1
D	18-3	18-3	18-3	18-7	18-7	18-8	18-8	18-8	18-8	18-9	18-9	10-11	10-7	9-7	9-7
RATE OF FLOW Gal / sec	.495	.482	.50	.486	.482	.477	.482	.482	.482	.459	.472				
REMARKS	Pumping restarted 6.30 pm. 27.11.70											Machine switched off			

depend only on the distance between the point of observation and the centre of the well. Ignoring the effect of recharge which will be very small over this local area, the general drawdown formula for a well in a confined aquifer is given by

$$s = \frac{Q_o}{2\pi kH_s} \ln \frac{R}{r} \qquad \text{(from Dupuit (15))}$$

where

s = drawdown of water table.

H_s = saturated thickness of aquifer.

R = integration constant determined from the boundary conditions.

r = distance to observation well.

k = coefficient of permeability.

In the immediate vicinity of a well in an aquifer of large extent for $r < 0{,}1R$ the above equation simplifies to

$$s = \frac{Q_o}{2\pi kH} \ln \frac{R_o}{r} \tag{10}$$

where R_o is the integration constant for the well face. For all points of observation R_o is now constant. Therefore, plotting the observed drawdown s on a linear scale against the distance r on a logarithm scale will give a straight line relationship

$$s = \frac{Q_o(2{,}3)}{2\pi kH} \log R_o - \frac{Q_o(2{,}3)}{2\pi kH} \log r \tag{11}$$

$$= w - x \log r$$

therefore

$$k = \frac{1,15}{x \pi H} Q_o \qquad (12)$$

and

$$\log R_o = \frac{w}{x}$$

The plot of s against log r for pumping at 28ft. (8,5m) is given in Fig.4.2 and the curve given was fitted by eye for standpipes A1, B1, C1, D1.

From Fig.4.2 then

w = 23ft. (7m)

x = 12

from Table 4.1 the volume pumped at steady state

Q_o = ,0088 cu.secs ($2,5 \times 10^{-4} m^3$/sec)

therefore

$$k = \frac{1,15\ (8,8 \times 10^{-3})}{12 \pi 10} = 2,68 \times 10^{-5} \text{ft/sec}$$

$$= 8,2 \times 10^{-4} \text{ cm/sec}$$

4.2.2 Pumping at 50ft. (15,2m).

From Tables 4.2 and 4.3 it can be seen that quite large drawdowns were achieved when pumping from this depth. Considerable variation in drawdown (approximately 6ft. (1,8m)) was noted in the standpipes at the furthest points from the well, i.e. A and D again emphasizing the variations in permeability across the site. The standpipes at B and C, however, showed comparable drawdown to 22ft. (6,7m) below ground level.

Perhaps the most significant point of these results is the large variation in drawdown between the standpipes at different depths, i.e. between B and Bl, and D and Dl. The standpipes located in the upper layers, i.e. Bl, Dl, etc., were almost unaffected by pumping from 50ft. (15,2m) which again indicates the existence of a leaky confined aquifer below 43ft. (13,1m) as suggested in section 4.2.1.

This analysis will be based on the assumption of pumping from a confined aquifer as was assumed in section 4.2.1. The aquifer in this instance is considered to extend between the boundaries of impervious layers at 43ft. (13,1m) and 85ft. (26m). The assumption of unidirectional flow made in the previous section is no longer valid here because the well no longer fully penetrates the aquifer and the curvature of the flow lines make it impossible to neglect the vertical components of flow. However, the deviation from a unidirectional flow pattern is limited to distances not greater than twice the saturated thickness from the well centre, and this local nature of the curvature allows its effect to

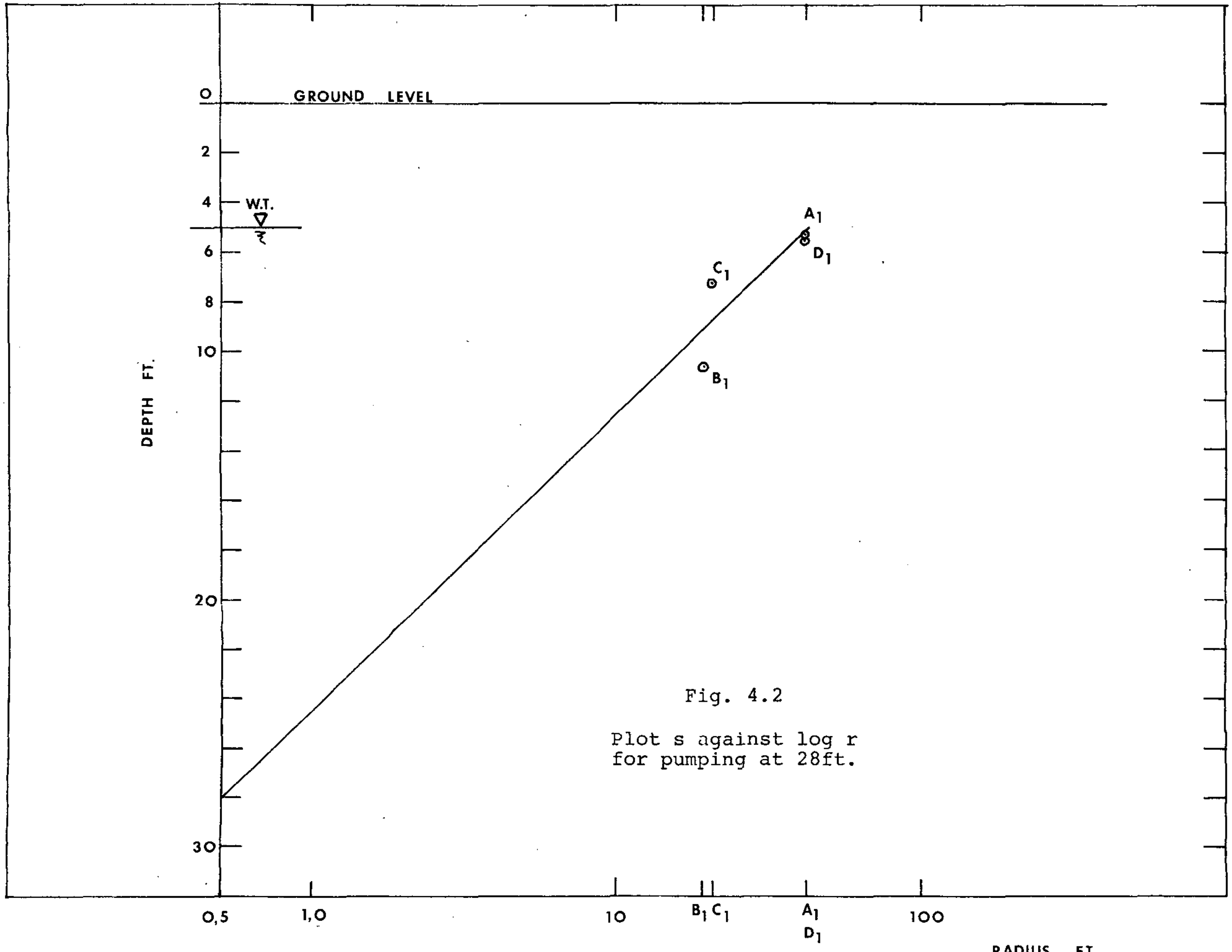


Fig. 4.2

Plot s against log r
for pumping at 28ft.

be calculated as a correction to the drawdown of small capacity, fully penetrating wells. The flow in partially penetrating wells is dependent upon the penetration P_H of the well screen into the aquifer and the drawdown equation for partially penetrating wells in a confined aquifer is given by

$$s = \frac{Q_o}{2\pi kHG_K} \ln \frac{R}{r}$$

where G_K is a correction factor for partial penetration and is equal to the ratio of flow from a partially penetrating well to that from a fully penetrating well for the same drawdown at the well. Kozeny (21) has developed the following equation for this correction factor

$$G_K = \frac{P_H}{H} \left(1 + 7\sqrt{\frac{r_o}{2P_H}} \cos \frac{\pi P_H/H}{2}\right)$$

where the notation is as previously defined. For the particular case of pumping at 50ft. (15,2m) G_K = 0,405. The plot of s against log r is given in Fig.4.3 and the best fit has been taken between the results from the standpipes at A, B, C and D.

The analysis is similar to section 4.2.1 and from equation (11) and allowing for the correction factor G_K we have

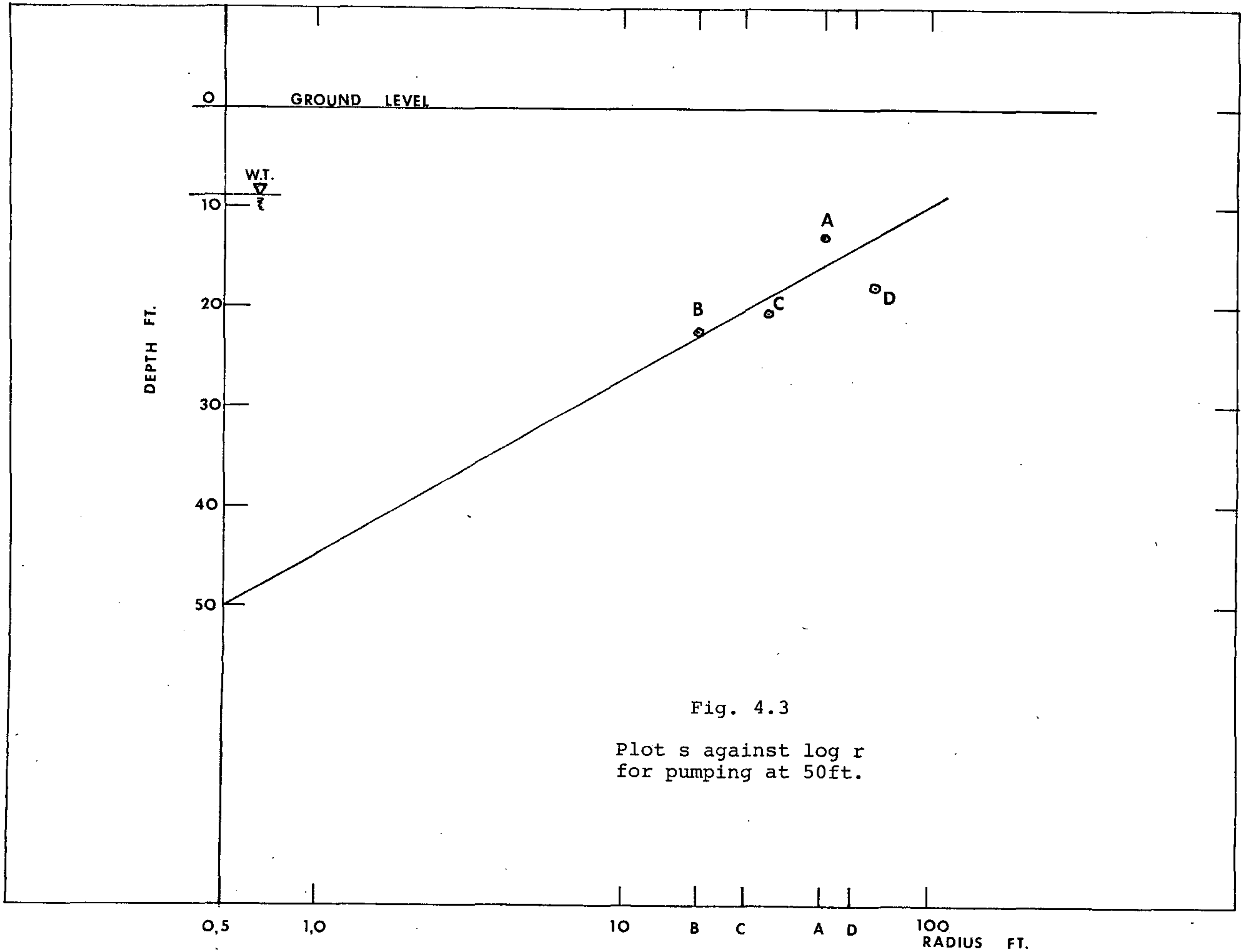


Fig. 4.3

Plot s against log r
for pumping at 50ft.

$$s = \frac{Q_o 2,3 \log R_o}{2\pi kHG_K} - \frac{Q_o(2,3) \log r}{2\pi kHG_K}$$

$$= w' - x' \log r$$

therefore

$$k = \frac{1,15\ Q_o}{b'\pi\ HG_K}$$

and from Fig 4.3

$w' = 41$ft.

$x' = 17,3$ ft.

From Table 4.3 the flow at steady state is recorded as

$Q_o = 0,076$ cu.secs $(2,15 \times 10^{-3} m^3/sec)$

therefore

$$k = \frac{1,15\ (0,076)}{17,3\pi 43\ (0,405)} = 9,24 \times 10^{-5} ft/sec$$

$$= 2,82 \times 10^{-3}\ cm/sec$$

4.2.3 Conclusions from pumping test.

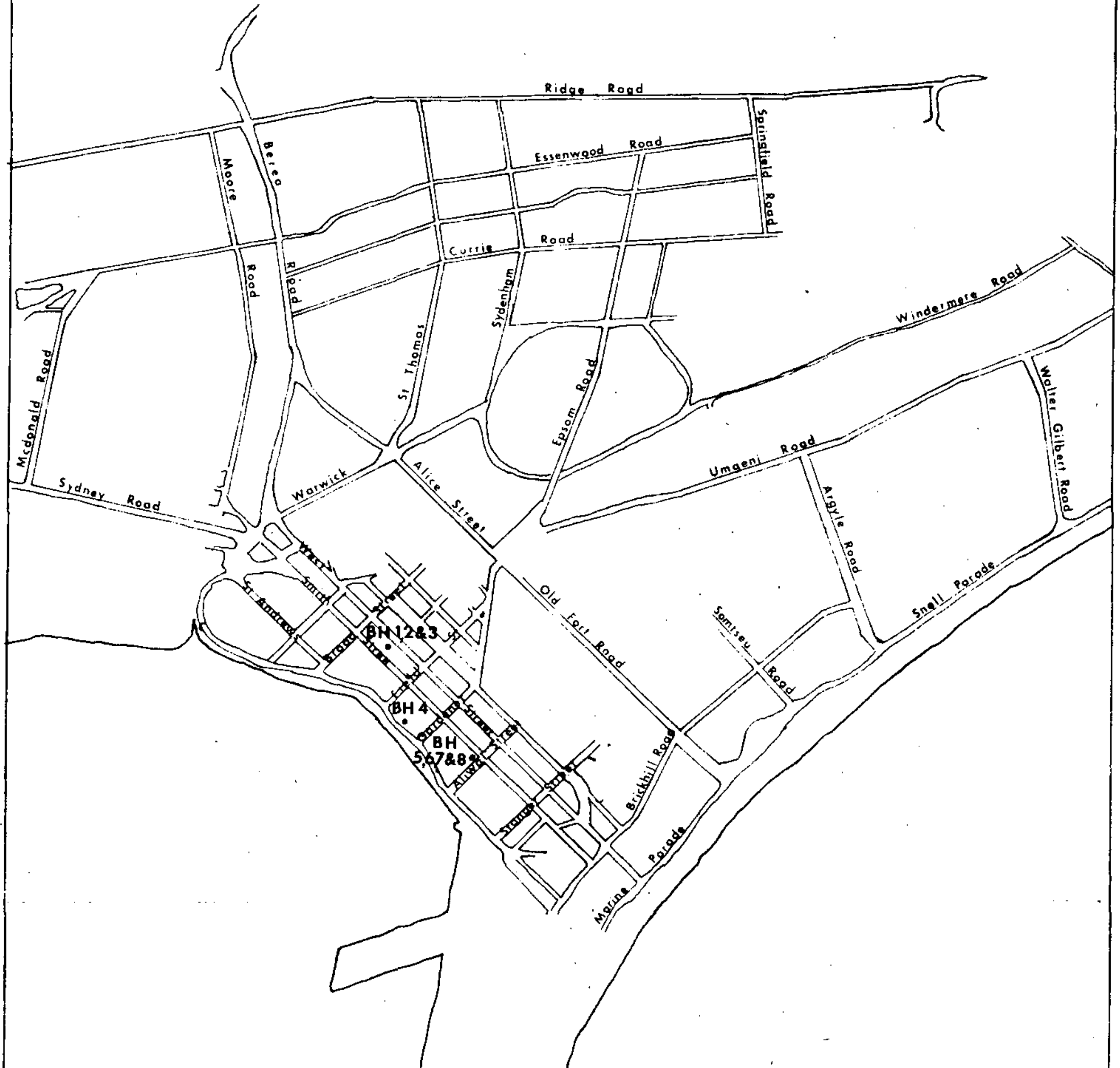
The extremely varied nature of the site can be seen from the borehole profiles BH.1-3 given in Appendix A and the results of the test pumping analysis should be viewed in the light of these variations. However, even though the tests were carried out at different levels and in a variable material, the difference in permeabilities (3 to 4 times) is not large.

The values obtained may therefore be assumed to be a reasonable indication of the order of magnitude of the average permeability for this site.

4.3 Permeability from laboratory tests on disturbed and undisturbed samples.

Appendix A contains a series of borehole logs taken at random sites in the central Durban area. The approximate positions of these boreholes are shown in Fig.4.4.

It can be seen from these boreholes that large variations in soil conditions occur throughout central Durban. In an attempt to estimate the permeability of the various layers, although these in themselves are not strictly consistent, samples both disturbed and undisturbed were extracted and tested in the laboratory. Undisturbed samples were recovered only from the more clayey and silty layers and thus the more reliable test results must of necessity relate to these more impermeable layers. However, these results in themselves are useful in that they give an indication of the lowest values of permeability which can be expected. This will give some guidance to the average permeability for the area if only in the capacity of a lower limit.



CITY OF DURBAN

BOREHOLE POSITIONS

FIG. NO. 4.4

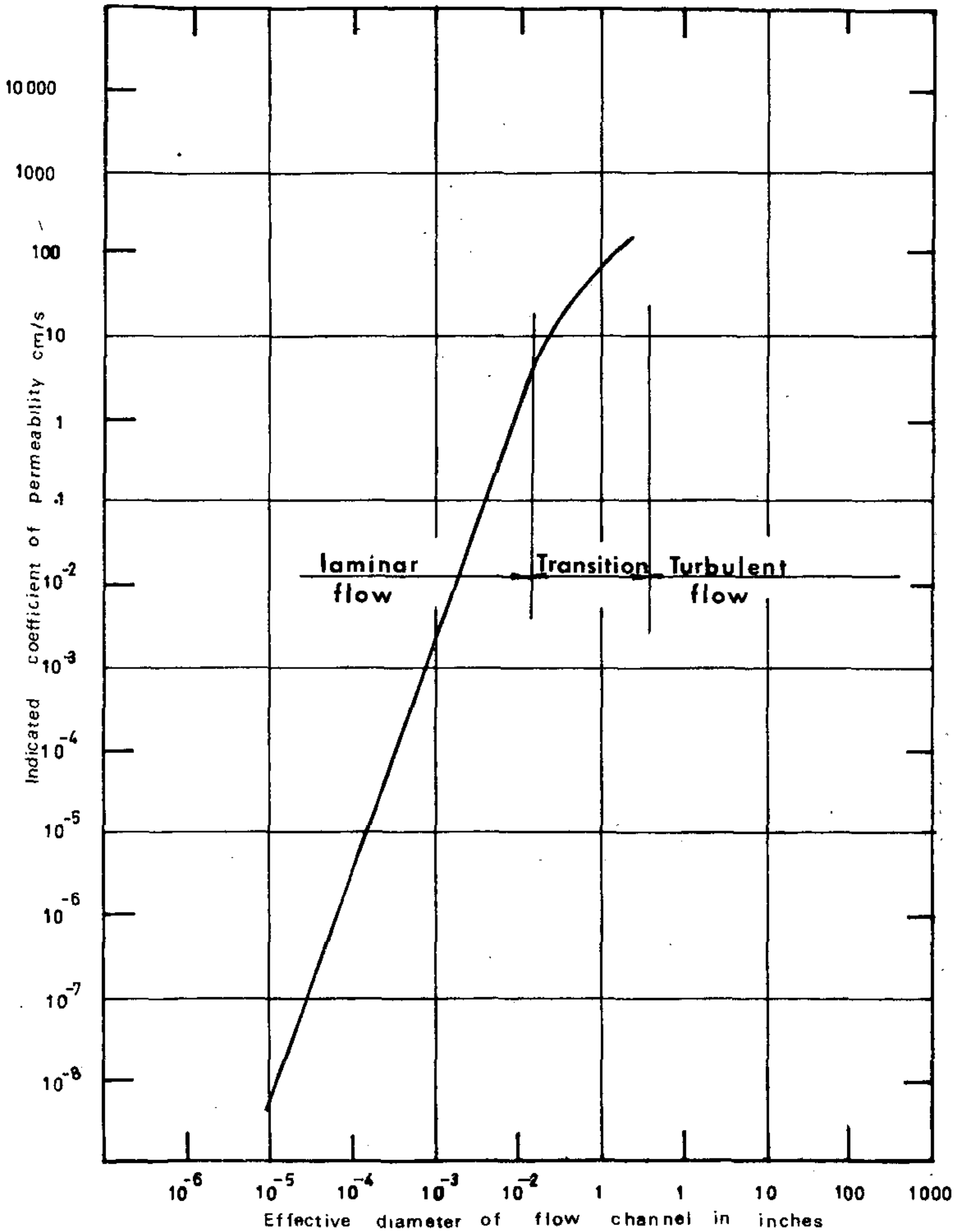
An indirect determination of permeability can be obtained from the particle size analysis.

From Newton's law of friction, it can be shown that permeability should vary approximately with the square of the sizes of pore spaces and particle sizes. The general validity of this approach has been verified by the testing of actual soils which indicate that the permeabilities of soils do increase at a rate slightly greater than the square of the pore diameters, (assuming pore diameter = $D_{10}/5$; where D_{10} is the grain size at which 10% by weight of the soil is finer).

A plot of coefficient of permeability versus effective diameters of flow channel is given in Fig.4.5 (6)

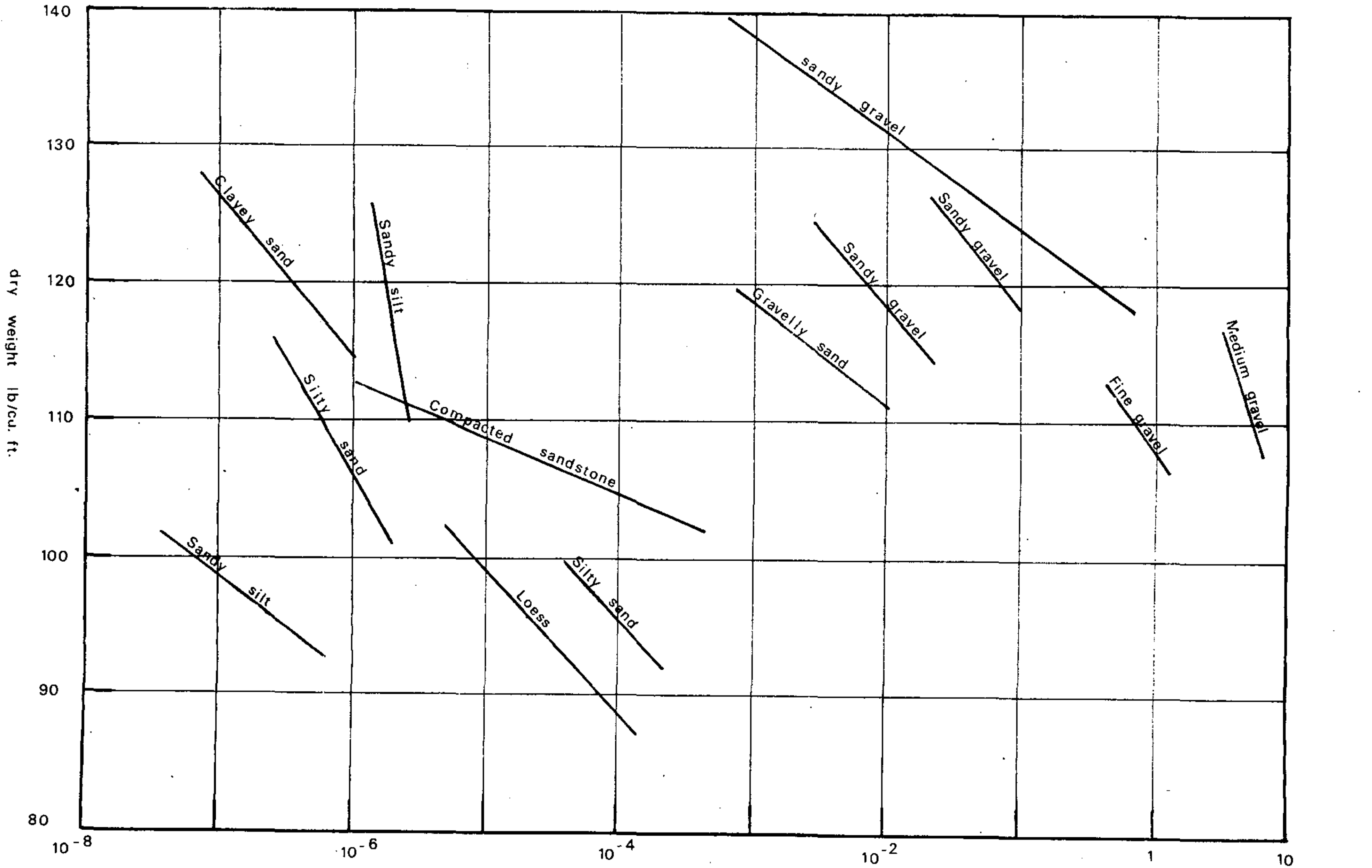
The influence of the density of the soil on its permeability provides another indirect method, and whilst it is less significant than grain size it can assist in confirming the order of magnitude of the coefficient of permeability obtained by other methods. Fig.4.6 gives a relationship between various soil types, density and coefficient of permeability. (6)

A number of samples were obtained from the boreholes and grading analyses and dry density determinations were carried out. Certain of these samples were undisturbed and were also



Variation in permeability with size of flow channel (after Cedergren)

Fig. 4.5



Relation between coefficient of permeability, soil type & density (after Cedegren)

Fig. 4.6

TABLE 4.4.

Indirect estimate of permeability.

BH No.	Depth of Sample ft.	Soil Type	Dry Density lb/ft^3	D10/5 inches	Permeability cm/sec	
					from fig. 4.5	from fig. 4.6
3 D	20	Clayey sand	111	4×10^{-4}	4×10^{-6}	4×10^{-6}
3 D	30	Silty sandy clay	101	$< 1 \times 10^{-5}$	$< 1 \times 10^{-9}$	$< 1 \times 10^{-8}$
3 D	40	Clayey silty sand	109	$< 1 \times 10^{-5}$	$< 1 \times 10^{-9}$	8×10^{-7}
5 D	45	Clay	77	$< 1 \times 10^{-5}$	$< 1 \times 10^{-9}$	$< 1 \times 10^{-8}$
5	83	Clayey silt	70	3×10^{-4}	6×10^{-6}	$< 1 \times 10^{-8}$
6 D	52	Silty sand	100	1×10^{-4}	1×10^{-6}	4×10^{-6}
6	85	Silty sand	77	7×10^{-4}	1×10^{-4}	$> 1 \times 10^{-4}$

Where D indicates that a constant head permeability test has been carried out on this sample. See Table 4.5.

used to determine permeability by the constant head method. By this means, the direct and indirect laboratory determinations could be compared. The results of the indirect laboratory tests are given in Table 4.4.

Undisturbed U4 samples were obtained from the boreholes at various layers as shown on the borehole profiles in Appendix A. These samples were tested by the constant head method to determine their permeabilities. The results are given in Table 4.5 below and may be compared with the values given in Table 4.4.

TABLE 4.5.

Direct measurement of permeability.

BH No.	Depth of sample ft. (m)	Soil Type	Permeability cm/s
3	20 (6,1)	clayey sand	$3,2 \times 10^{-7}$
3	30 (9,1)	silty sandy clay	$6,6 \times 10^{-8}$
3	40 (12,2)	clayey silty sand	$5,9 \times 10^{-5}$
5	45 (13,7)	clay	$7,1 \times 10^{-10}$
6	52 (15,9)	silty sand	$1,4 \times 10^{-7}$
7	42 (12,8)	clayey sand	$1,5 \times 10^{-6}$
8	52 (15,9)	clayey sand	$5,4 \times 10^{-8}$

From a comparison of the permeabilities for the samples given in Tables 4.4 and 4.5, it can be seen that a reasonably close relationship exists between both indirect and direct methods of estimating permeabilities for individual samples. The greatest variation occurs in the predominantly sandy samples as may be expected, and this is due to the difficulty in obtaining representative 'undisturbed' samples.

The indirect method, however, should only be used for confirmation of other methods due to the large number of variables which cannot be taken into account. The permeabilities of the sample at 52ft. (15,9m) in BH.6, for example, may appear fairly close in relation to the total range of permeabilities (10^{10} times) but they nevertheless represent a variation of approximately 10 times. The difference in permeabilities for the sample at 40ft. (12,2m) in BH.3 is approximately 80 times. In both cases the indirect method is probably in error.

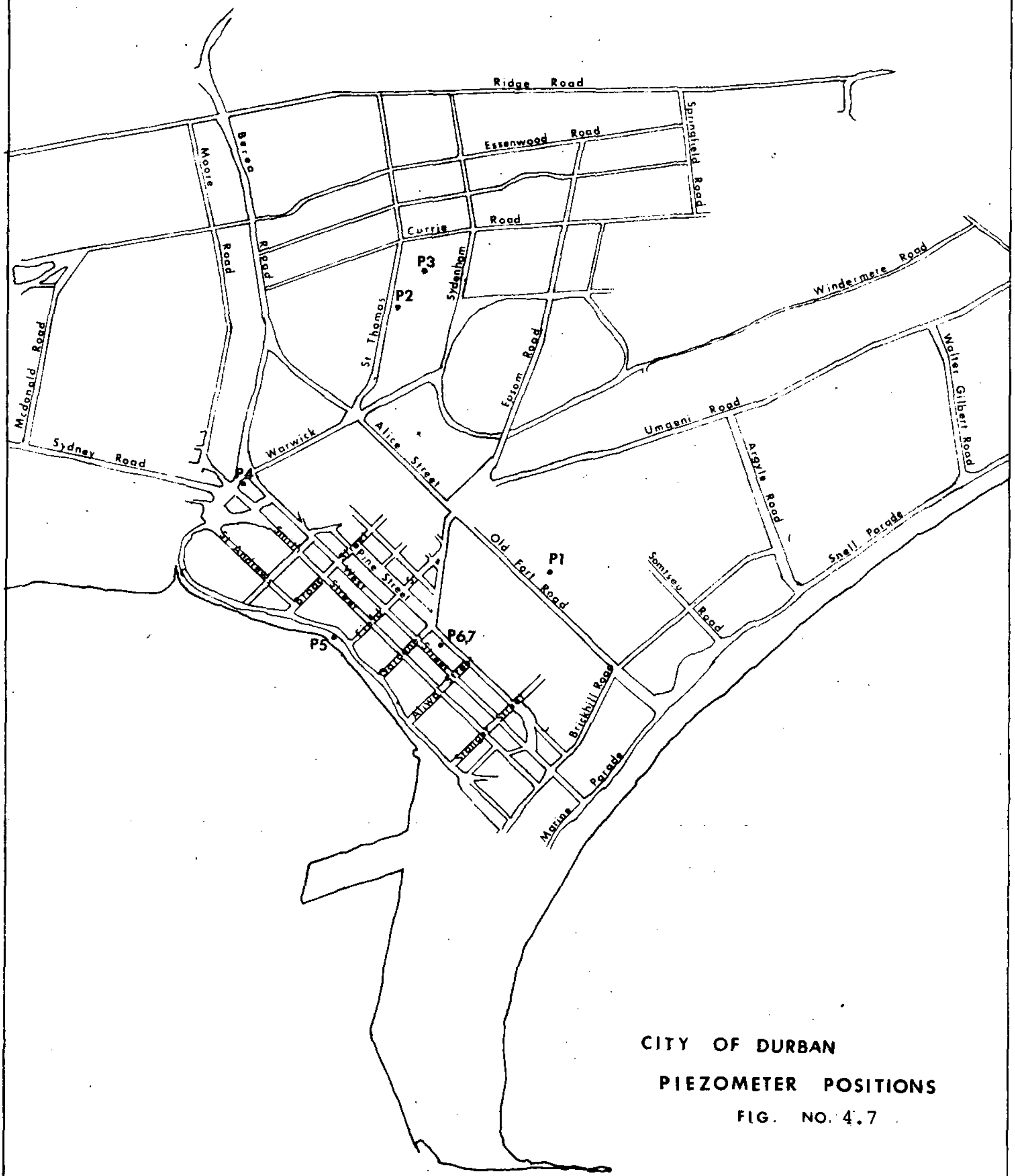
From the samples tested, a reasonable lower limit of permeability in the central area of Durban is 1×10^{-7} cm/sec. Where clay layers exist, this reduces to approximately 1×10^{-10} cm/sec, but due to the discontinuity of these very impermeable clay layers above 75 - 80ft. (22,8 - 24,4m) they will not significantly affect an average permeability for the area. They should, however, be considered in detail on individual sites where their presence or absence can significantly affect construction.

4.4 Permeability from piezometers.

During March, 1973, a series of piezometers were installed at selected positions throughout Durban. The locations were selected with a view to obtaining maximum information, but at the same time, obtaining positions which would be free from interference, thus permitting long term observations to be available. The locations of the piezometers are shown in Fig.4.7 and further information is given in Table 4.6.

Piezometer Nos. P6 and P7 were positioned approximately 600mm apart and at different depths in an attempt to determine the existence of any confined aquifers in this area.

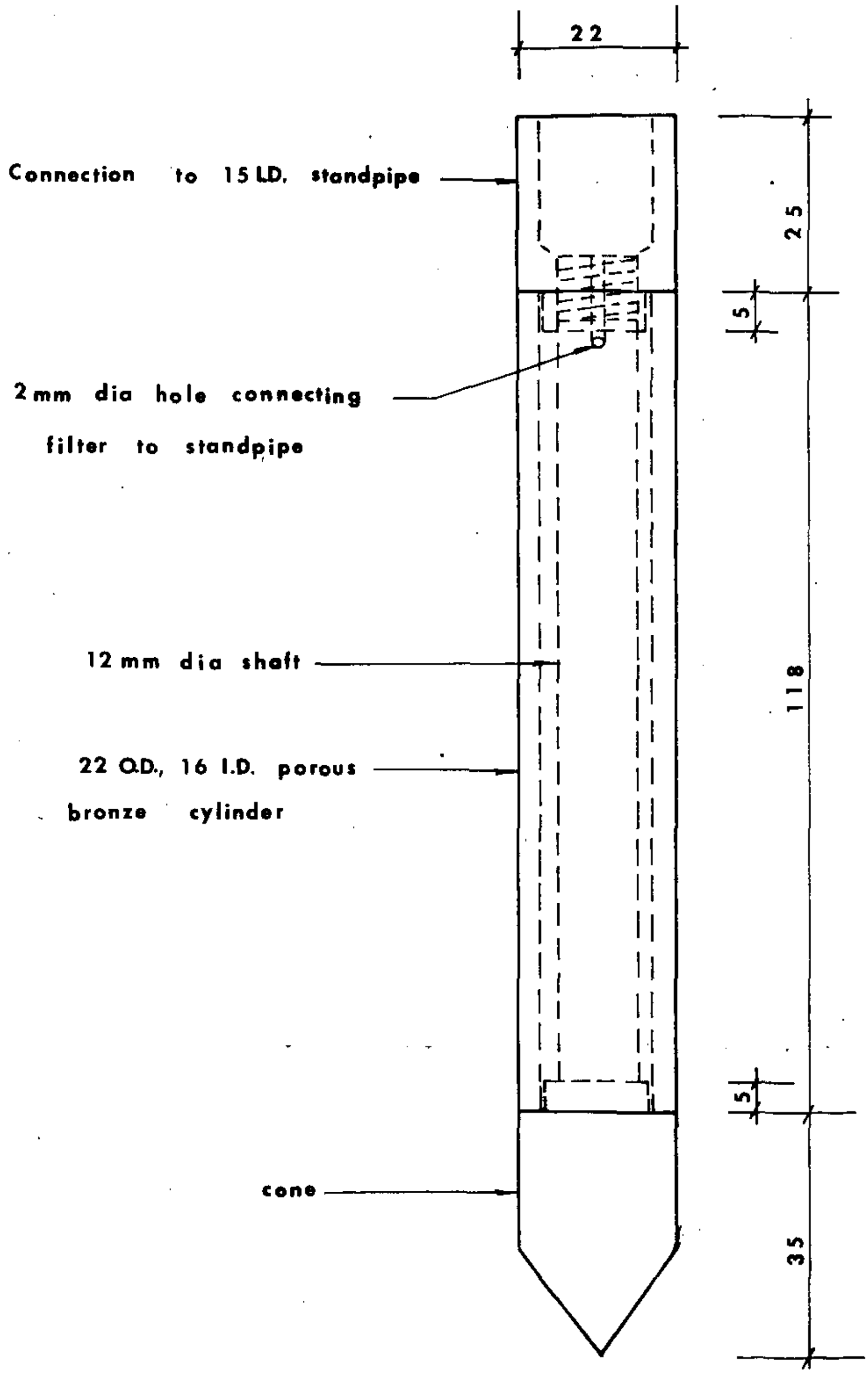
The piezometer, which is shown in Fig.4.8, consisted of a hollow porous bronze cylinder 118mm in length, and 22mm diameter, closed at its end by a bronze cone. Connected to the cone was a solid bronze shaft which passed through the porous bronze cylinder and was threaded at its end to take a standard 15mm I.D. water pipe. The water pipe was coupled at 3m centres with flush watertight couplings.



CITY OF DURBAN

PIEZOMETER POSITIONS

FIG. NO. 4.7



EXPERIMENTAL PIEZOMETER Fig. 4.8.

TABLE 4.6.

Information on Piezometers

Piezometer No.	Location	Date installed	R.L. of top of cover	R.L. of piezometer tip	R.L. of water table
P1	Old Fort Gardens	20th February, 1973	+ 7,14m	- 1,96m	+ 3,39m
P2	Botanic Gardens	20th February, 1973	+15,04m	+11,64m	+14,59m
P3	Botanic Gardens	27th March, 1973	+31,55m	+13,85m	+17,51m
P4	Natal Technical College	23rd February, 1973	+ 6,55m	- 2,39m	+ 4,85m
P5	Opposite to Point Yacht Club, Esplanade	26th February, 1973	+ 2,87m	- 3,74m	+ 1,23m
P6	Medwood Gardens	27th February, 1973	+ 6,27m	- 0,20m	+ 3,50m
P7	Medwood Gardens	28th February, 1973	+ 6,48m	- 3,86m	+ 3,72m

The detailed positions of these piezometers may be obtained from Professor Knight, Department of Civil Engineering, University of Natal, Durban, or from the author.

The most satisfactory method of installation was arrived at by trial and error before installation of the test piezometers. This was eventually found to be by advancing first, standard Dutch probe penetration test outer rods sealed at the bottom with a C.P.T. cone. When the cone reached the required depth, the piezometer tip and standpipe were lowered down the inside of the outer rods until the piezometer rested on the cone. The outer rods were then withdrawn. Before installation, the insides of the water pipe were cleaned by pulling lengths of clean rag attached to a wire lead through the pipe.

As a result of the high ground resistance to driving, the Dutch probe rods at the position of piezometer No. P3 could not be driven below the water table. The Dutch probe rods were therefore withdrawn and the piezometer was positioned as follows. N.W. rods were driven using a S.P.T. hammer to below the expected level of the water table. The piezometer tip and standpipe were then lowered inside the rods and the rods were then withdrawn. As a result of the expected high disturbance of the ground around the piezometer tip the variable head result for P3 is considered to be suspect.

The water level was measured using an 'electrical dipper' which consisted of a pair of insulated wires from a double wire cable connected in series to a battery and galvanometer. The end 5mm of the wires were uninsulated and from 5mm to 150mm from the end of the wire was a short length of narrow steel tubing which acted as a weight. The wires were lowered down the standpipe, and when the short uninsulated ends came into contact with the water surface the galvanometer was activated. Records of observed water level in the piezometers are given in Tables 4.7 to 4.9.

The small difference in levels recorded in piezometer Nos. P6 and P7 is indicative of a confined aquifer between levels -0,2m and -3,86m. This is to be expected in the multi layered deposits which exist in the Durban area and which are shown in the bore-holes in Appendix A.

After the water level had settled down in the standpipes, insitu variable head permeability tests were carried out on piezometers P1, P3, P4, P5, P6 and P7.

The standpipe was slowly filled with water to avoid causing excessive air bubbles. As the water level fell the standpipe was continuously topped up over a period of 10-15 minutes to allow any air bubbles which did occur to disperse. Readings were then taken of the depths to the water level in the standpipe. The times at which the readings were taken and the corresponding heads of water in the standpipes are given in Tables 4.10 to 4.12.

The field variable head determination follows

TABLE 4.7.

Water Table Observations

Piezometer No.	Date	Time	R.L. of water table
P1	20:2:73	1430	- 1,06
	21:2:73	0730	+ 1,14
		1605	+ 1,63
	22:2:73	1000	+ 2,55
	1:3:73	1300	+ 3,37
	14:3:73	0830	+ 3,43
	21:3:73	0900	+ 3,39
	24:3:73	0925	+ 3,38
	31:3:73	1155	+ 3,39
P2	20:2:73	1700	+12,04
	21:2:73	0700	+14,14
		1630	+14,19
	22:2:73	1020	+14,41
	23:2:73	0720	+14,57
	1:3:73	1230	+14,59
	14:3:73	0800	+14,59
	21:3:73	0805	+14,58
	31:3:73	1045	+14,59
P3	28:3:73	0800	+16,87
	30:3:73	0740	+17,50
	31:3:73	1020	+17,47
	6:4:73	1500	+17,52
	26:5:73	0830	+17,51

TABLE 4.8.

Water Table Observations

Piezometer No.	Date	Time	R.L. of water table
P4	23:2:73	1620	+ 2,90
	1:3:73	1200	+ 4,87
	14:3:73	0845	+ 4,84
	21:3:73	0845	+ 4,85
	4:4:73	1330	+ 4,85
P5	26:2:73	1615	- 3,15
	1:3:73	1330	+ 1,01
	14:3:73	1400	+ 1,02
		1615	+ 1,22
	20:3:73	1745	+ 1,20
	21:3:73	1045	+ 1,24
		1640	+ 1,24
	23:3:73	1555	+ 1,24
	28:3:73	1215	+ 1,16
	31:3:73	1245	+ 1,14
	3:4:73		+ 1,19
	4:4:73	1420	+ 1,22
	5:4:73	1120	+ 1,29
		1730	+ 1,33
	7:5:73	1130	+ 1,32
	6:8:73	1100	+ 1,31
	13:8:73	1535	+ 1,33
	21:8:73	1600	+ 1,31

TABLE 4.9.

Water Table Observations

Piezometer No.	Date	Time	R.L. of water table	
P6	27:2:73	1430	+ 0,11	
	1:3:73		+ 1,90	
	14:3:73	1415	+ 3,46	
	20:3:73	1100	+ 3,49	
	23:3:73	1610	+ 3,49	
	24:3:73	1100	+ 3,50	
	28:3:73	1425	+ 3,49	
	29:3:73	0815	+ 3,50	
	7:5:73	1145	+ 3,47	
	6:8:73	1100	+ 3,43	Heavy rain for 2 days prior
	13:8:73	1600	+ 3,61	
	21:8:73	1545	+ 3,57	
P7	28:2:73	1005	- 3,86	
	1:3:73		- 1,94	
	14:3:73	1420	+ 3,53	
	20:3:73	1105	+ 3,66	
	23:3:73	1615	+ 3,69	
	24:3:73	1100	+ 3,71	
	29:3:73	0815	+ 3,73	
	7:5:73	1150	+ 3,72	
	6:8:73	1100	+ 3,65	Heavy rain for 2 days prior
	13:8:73	1600	+ 3,78	
	21:8:73	1550	+ 3,79	

TABLE 4.10

Variable Head Field Permeability Test

Piezometer No. P1.

Time t		ht = head at time t	$^{ht}/_{h_o}$
24:3:73	0940	3,61m	1,0
	0941	3,57	0,99
	0943	3,49	0,97
	0947	3,35	0,93
	0955	3,09	0,86
	1010	2,65	0,74
	1025	2,28	0,63
	1040	1,96	0,54
	1215	0,76	0,21
	1410	0,26	0,07

Piezometer No. P3.

Time t		ht = head at time t	$^{ht}/_{h_o}$
26:5:73	0835	13,95m	1,0
	0850	13,43	0,96
	0905	12,92	0,93
	1000	11,47	0,82
	1205	8,87	0,64
	1710	5,45	0,39

TABLE 4.11

Variable Head Field Permeability Test

Piezometer No. P4.

	Time t	ht = head at time t	ht/h_o
4:4:73	1335	1,70m	1,0
	1336	1,43	0,84
	1337	1,16	0,69
	1339	0,83	0,49
	1343	0,47	0,28
	1350	0,16	0,09
	1355	0,09	0,05

Piezometer No. P5.

	Time t	ht = head at time t	ht/h_o
4:4:73	1430	1,65m	1,0
	1431	1,47	0,89
	1432	1,38	0,84
	1434	1,17	0,71
	1438	0,86	0,52
	1445	0,52	0,31
	1450	0,39	0,23
	1455	0,29	0,17
	1500	0,22	0,13

TABLE 4.12

Variable Head Field Permeability Test

Piezometer No. P6.

	Time t	ht = head at time t	ht/h_o
24:3:73	1122	2,77m	1,0
	1123	2,71	0,98
	1124	2,63	0,95
	1126	2;51	0,91
	1130	2,27	0,82
	1138	1,88	0,68
	1154	1,27	0,46
	1220	0,66	0,24
	1420	0,06	0,022

Piezometer No. P7.

	Time t	ht = head at time t	ht/h_o
29:3:73	0825	2,75m	1,0
	0826	2,12	0,77
	0827	1,64	0,60
	0833	0,67	0,24
	0835	0,52	0,19
	0840	0,35	0,13
	0845	0,24	0,09
	0850	0,17	0,06

Hvorslev's method (1951) (6) U.S. Corps of Engineers, Waterways Experiment Station, using the Basic Time Lag determination and assuming:

a. Infinite depth and directional isotropy (k_v and k_h constant).
b. No disturbance, segregation swelling or consolidation of the soil.
c. No sedimentation or leakage.
d. No air or gas in the soil or piezometer.
e. Hydraulic losses in pipes and piezometer negligible.

The method of installation does not strictly comply with assumption b. above, but the amount of disturbance and consolidation was kept to the minimum possible and should not significantly affect the results obtained.

When a piezometer is installed, it is unlikely that the initial hydrostatic pressure in the piezometer will equal the true pore water pressure in the surrounding soil. Water must therefore flow to or from the piezometer until these pressures are equalized. The flow which occurs must have a corresponding time lag when the surrounding pore pressure varies.

The magnitude of the time lag is inversely proportional to the permeability of the soil and proportional to the size and type of pressure measuring device.

If water flows out of a casing the flow q is given by $q = Fkh = Fk\ (z_o - y)$

where

F = shape factor dependent upon shape and size of well

h = active head cm

z_o = distance from piezometric level to reference level cm

y = distance from piezometric level to reference level in the transient state cm (see Fig.4.9)

Considering the flow during time dt and ignoring friction losses in the pipe

$$q\ dt = A\ dy \tag{14}$$

where A = cross sectional area of standpipe (cm^2)

therefore from

$$\frac{dy}{z_o - y} = \frac{Fk}{A}\,dt \tag{15}$$

The volume required to equalize the pressure difference is

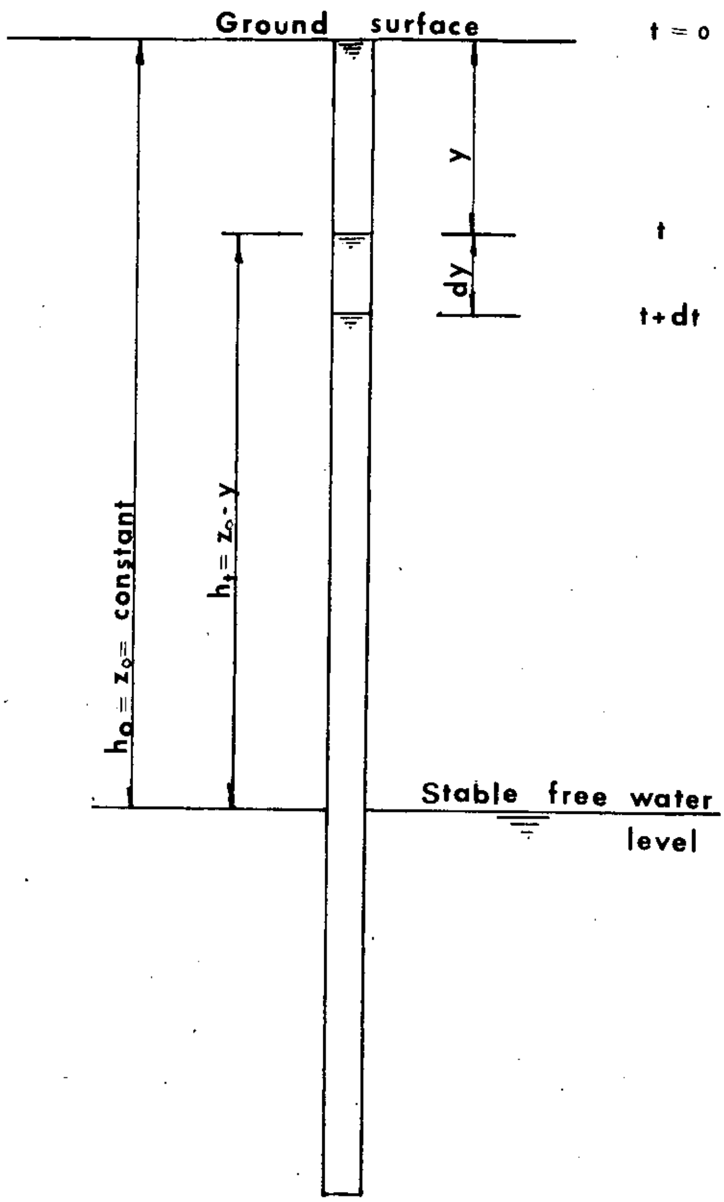
$$V = Ah$$

if the Basic Time Lag T is defined as the time required for equalization of the pressure difference when the original rate of flow $q = Fkh$ is maintained, then:

$$T = \frac{V}{q} = \frac{Ah}{Fkh} = \frac{A}{Fk} \tag{16}$$

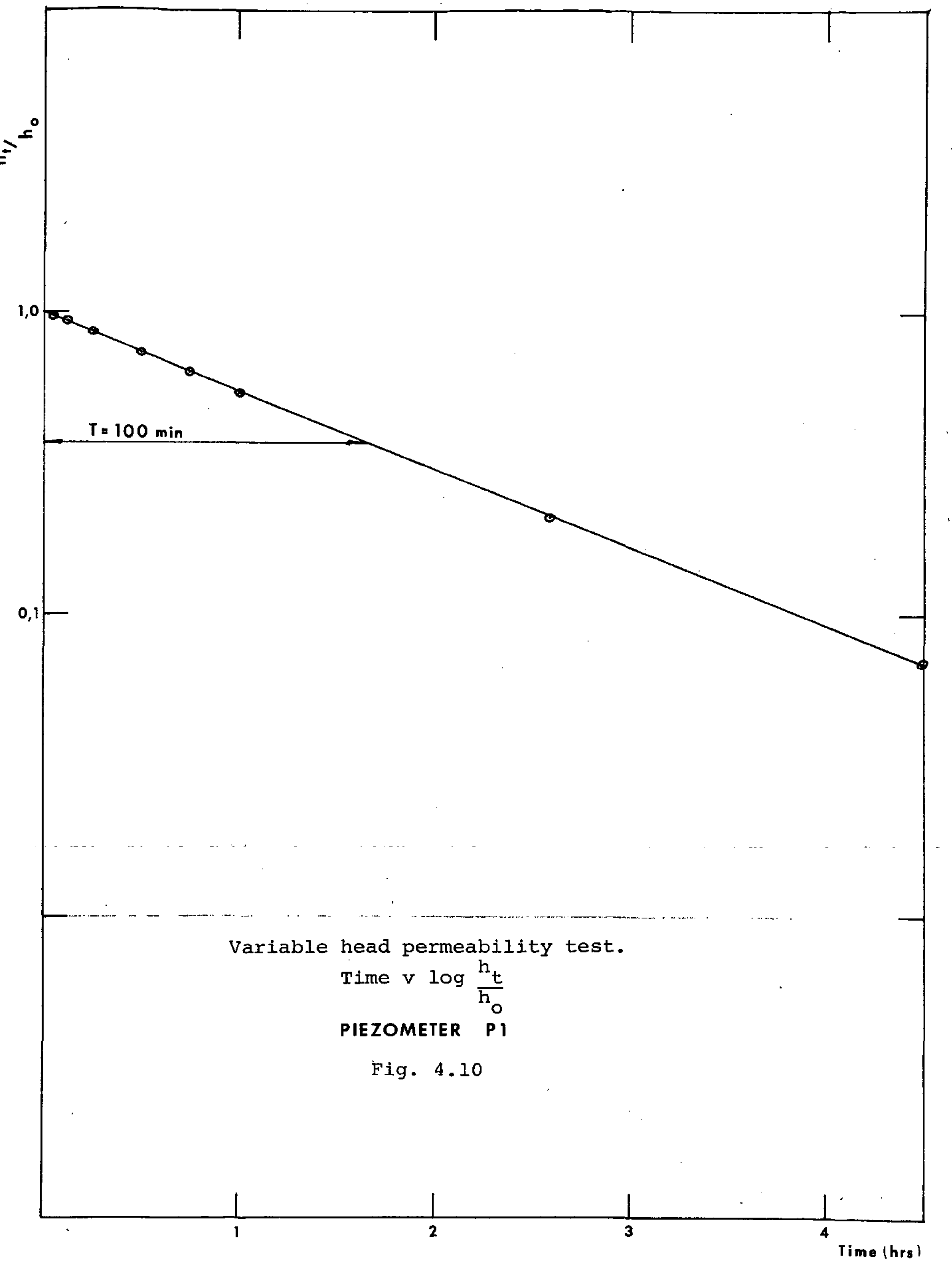
therefore $k = \frac{A}{TF}$

From the variable head field tests a plot of time on arithmetic scale versus head ratio h/ho on a log scale is made. The Basic Time Lag is the time at



Conditions for time lag determination
(After Hvorslev, U.S. Corps of Engineers, W.E.S.)

Fig. 4.9

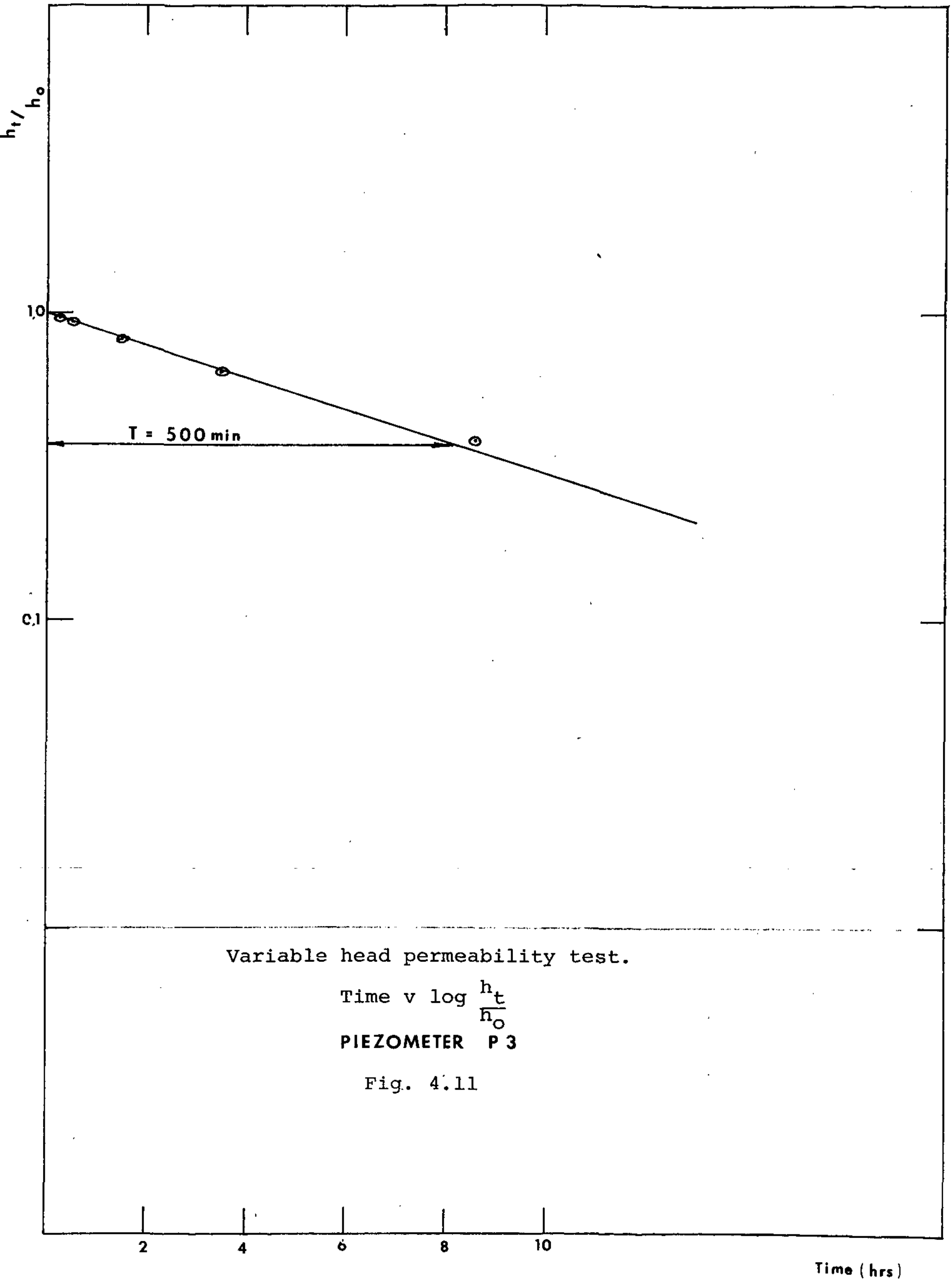


Variable head permeability test.

Time v log $\frac{h_t}{h_o}$

PIEZOMETER P1

Fig. 4.10

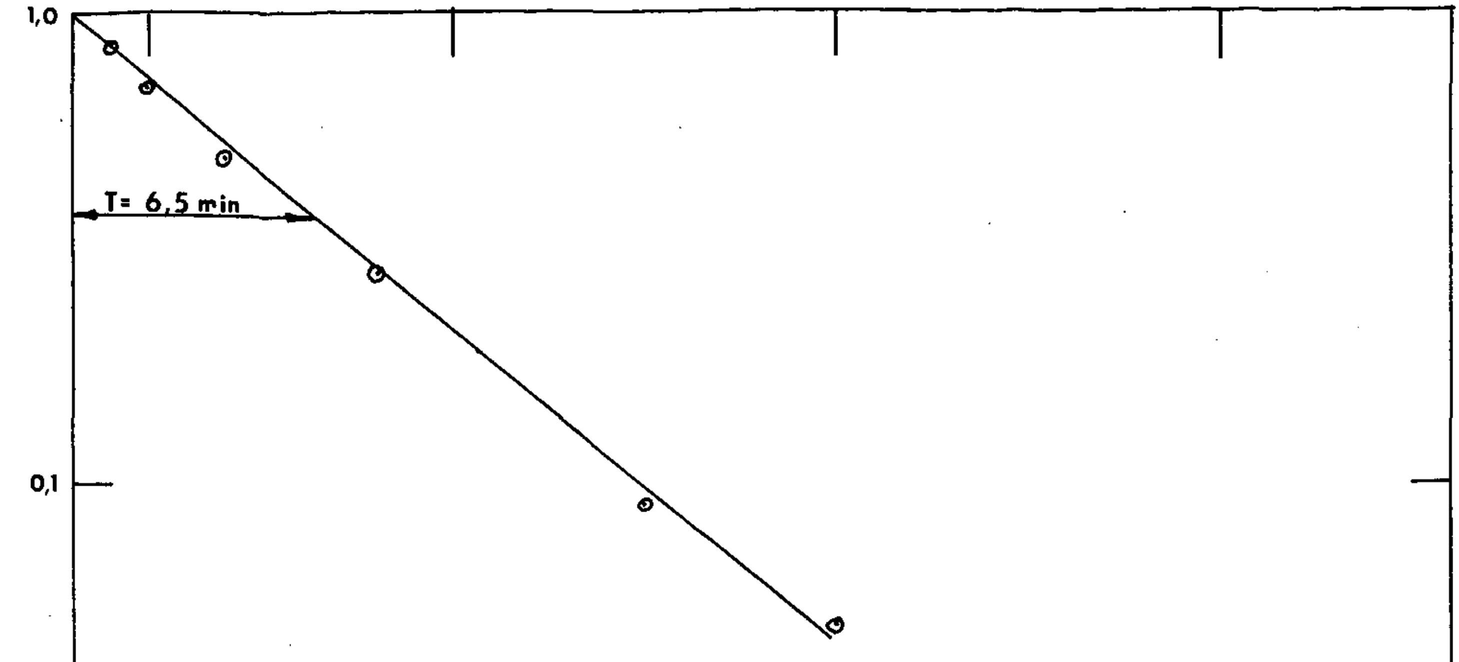


Variable head permeability test.

Time v log $\frac{h_t}{h_o}$

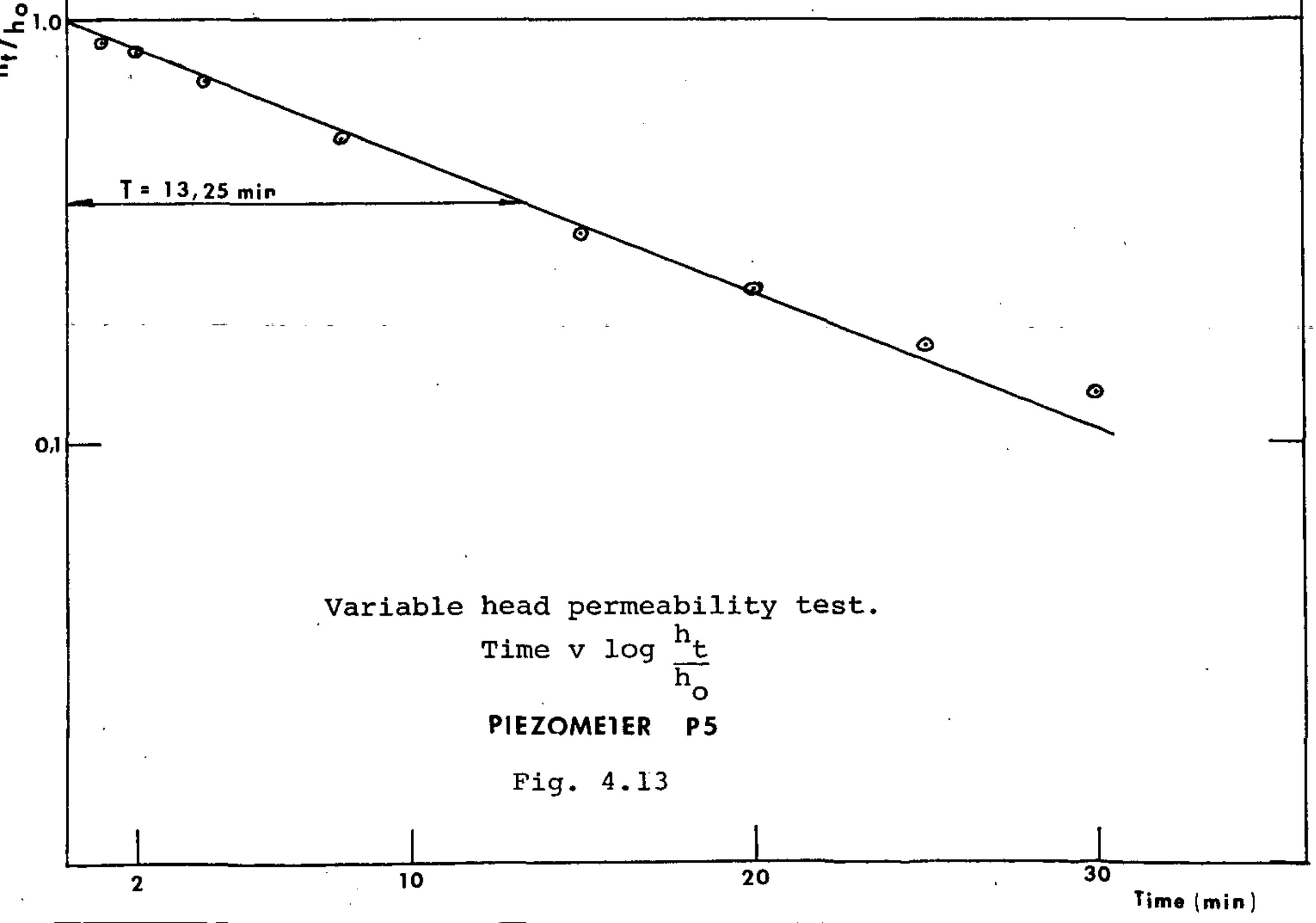
PIEZOMETER P 3

Fig. 4.11



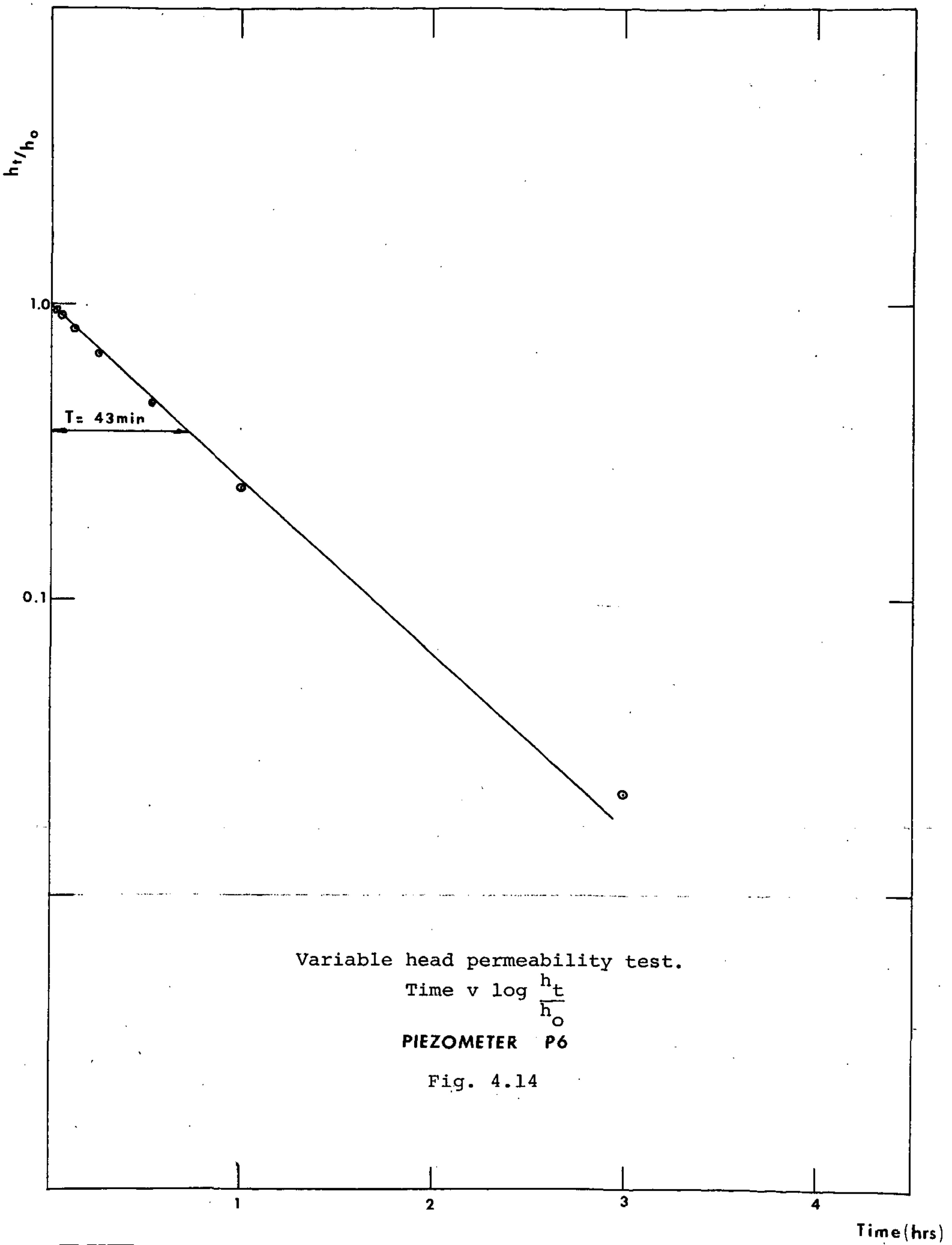
Variable head permeability test.
Time v log $\frac{h_t}{h_o}$
PIEZOMETER P4

Fig. 4.12



Variable head permeability test.
Time v log $\frac{h_t}{h_o}$
PIEZOMETER P5

Fig. 4.13

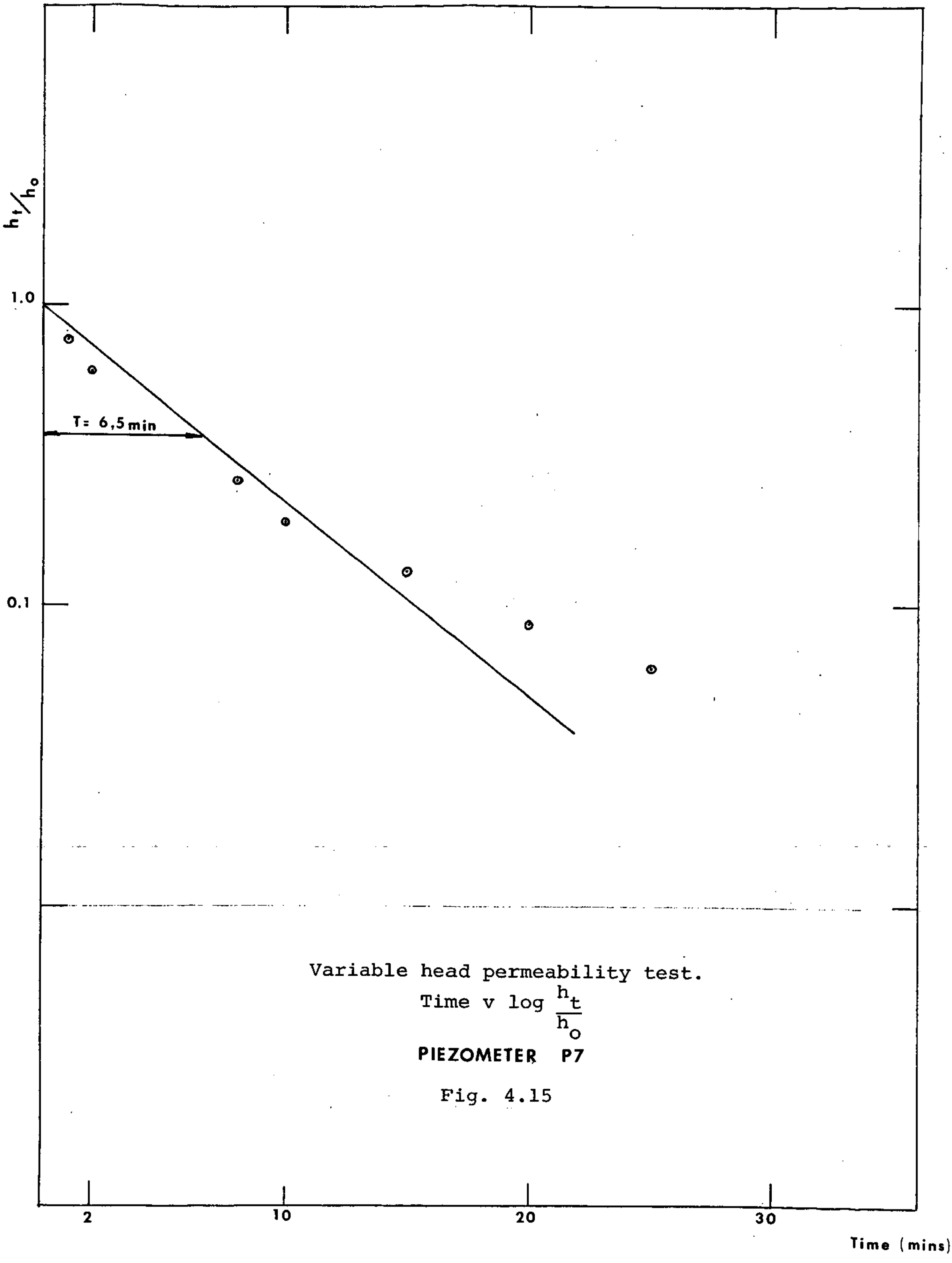


Variable head permeability test.

Time v log $\frac{h_t}{h_o}$

PIEZOMETER P6

Fig. 4.14



Variable head permeability test.

Time v log $\frac{h_t}{h_o}$

PIEZOMETER P7

Fig. 4.15

which the head ratio equals 0,37.

Assuming the value of the shape factor F from Hvorslev (6)

$$F = \frac{2\pi L_s}{\ln\left(\frac{2mL_s}{D}\right)} \quad \text{for } \frac{mL_s}{D} > 4$$

where

L_s = length of intake sample cm

m = transformation ratio = $\sqrt{k_h/k_v}$ assumed equal to unity

D = diameter of intake sample cm

$$\text{then } k = \frac{d^2 \ln\left(\frac{2mL_s}{D}\right)}{8\, L_s T} \qquad (17)$$

where

d = diameter of standpipe cm

The plots of time versus log h/h_o for the tests are given in Figs. 4.10 to 4.15 and permeability results based on equation (17) in Table 4.13.

TABLE 4.13.

Summary of Variable Head

Field Permeability Test.

Piezometer No.	R.L. of Piezometer tip m.	Permeability* cm/s
P1	- 1,96	$9,4 \times 10^{-6}$
P3	+13,85	$1,88 \times 10^{-6}$
P4	- 2,39	$1,45 \times 10^{-4}$
P5	- 3,74	$7,0 \times 10^{-5}$
P6	- 0,20	$2,19 \times 10^{-5}$
P7	- 3,86	$1,45 \times 10^{-4}$

* Assumes $k_h = k_v$

4.5 Average permeability.

From sections 4.1 to 4.4 an estimate of an average value of the coefficient of permeability may be made.

4.5.1 From section 4.1 the best approximation is $2,4 \times 10^{-2}$ cm/s.

4.5.2 From section 4.2 the estimated values are $8,2 \times 10^{-4}$ cm/s at 28ft. (8,5m) and $2,82 \times 10^{-3}$ cm/s at 50ft. (15,2m).

4.5.3 From section 4.3 a lower limit of 1×10^{-7} cm/s is indicated.

4.5.4 The field permeability tests given in section 4.4 tend to show a variation of coefficient of permeability between $1{,}45 \times 10^{-4}$ cm/s and $9{,}4 \times 10^{-6}$ cm/s.

The lower value at P3 is discounted due to disturbance of the soil during installation.

On the basis of the above, an average coefficient of permeability may be estimated as follows:-

It can be easily shown that the flow through a multi-layered system is given by :

$qh = k_1 i\ H_1 + k_2 i\ H_2 + k_3 i\ H_3 + \ldots\ldots$ etc.

where

qh = total flow

$k_{1,2,3\text{ etc.}}$ = coefficient of permeability of layer 1,2,3 etc.

i = hydraulic gradient

$H_{1,2,3\text{ etc.}}$ = thickness of layer 1,2,3 etc.

and the total flow qh is also given by

$qh = k_a i\ H$

where k_a = average permeability

H = total thickness of all layers ($H_1 + H_2 + H_3$ etc.)

therefore

$$k_a = \frac{1}{H}(k_1H_1 + k_2H_2 + k_3H_3 + \ldots\ldots \text{ etc.})$$

From a study of the soil profiles in BH Nos. 1-8 it is difficult to approximate on the occurrence of the various soil types and their permeabilities. However, in conjunction with section 4.5.1 to 4.5.4 above, assume that

40% of the material has permeability of the order of 1×10^{-3} cm/s and 60% a value of 1×10^{-7} cm/s; therefore the average permeability k_a is given by

$$k_a = \frac{1}{H}(1 \times 10^{-7} \times 0{,}6H + 1 \times 10^{-3} \times 0{,}4H)$$
$$= 4 \times 10^{-4} \text{ cm/sec.}$$

5.0 Effect of dewatering.

An attempt has been made to determine if a general dewatering problem exists. No attempt has been made to study individual sites and local effects due to the variable nature of both site construction methods and of the soil. Each site will require a separate investigation but in the light of the assumption made here, with respect to soil, pumping conditions and recharge, an assessment of the order of magnitude of any particular site problem may be made.

5.1 Effect on flow conditions.

The effect of dewatering a 'typical' excavation is investigated in order to determine approximately the quantity of water removed.

The general equation for drawdown ($H_s - h_d$) at any point caused by pumping a group of gravity wells is (ref. 21)

$$H_s^2 - h_d^2 = \frac{1}{\pi k} \sum_{i=1}^{i=n_g} Qw_i \ln \frac{R_i}{r_i}$$

where Qw_i = discharge from i^{th} well.

R_i = radius of influence of i^{th} well.

r_i = distance from i^{th} well to position at which drawdown is calculated.

n_g = number of wells in the group.

For a rectangular group of wells let

$2b_1$, $2b_2$ = long and short side length respectively.

h_c = head at centre of group.

h_w = head at well.

H_s = original head.

then

$$H_s^2 - h_c^2 = \frac{1}{\pi k} . n_g \; Qw \ln \frac{R_g}{\frac{4}{\pi}\sqrt{b_1 b_2}}$$

and

$$H_s^2 - h_w^2 = \frac{1}{\pi k} . Qw \ln \frac{R_g^{\,n}}{n_g r_w \; \frac{4}{\pi}\sqrt{b_1 b_2}^{\,(n-1)}}$$

where R_g = radius of influence of the well group.

Qw = discharge per well.

For an excavation 40m x 30m x 12m deep where H_s = 28m, k = 4 x 10^{-4} cm/s and the water table is 2m below ground level, let

h_c = 17m

n_g = 18

R_i = 100m (ref. pumping test section 4.2)

then

$$Qw = \frac{(28^2 - 17^2)\ \pi\ (4 \times 10^{-4})}{18 \times 100 \times \ln \frac{100}{\frac{4}{\pi}\sqrt{300}}}$$

$$= 2,28 \times 10^{-4} \ m^3/sec$$

$$= 19,7 \ m^3/day$$

total discharge from excavation

$$= 18 \times 19,7 \doteqdot 355 \ m^3/day$$

This is approximately 15 percent of the total estimated daily recharge, and whilst the removal of this quantity of water is unlikely to have serious consequences in terms of flow patterns it should be recalled that this is a very approximate estimate based on imperfect data.

The result is very sensitive to changes in permeability and seasonal or annual reductions in recharge, which could significantly increase the percentage of the recharge extracted. Under such conditions, the de-watering of a number of deep excavations in the Durban central area could result in the removal of quantities of water close to or in excess of the daily recharge with consequent changes in the flow paths of inflowing water.

The change in flow would consist of two separate effects:

a. There would be a change in the direction of the existing flow from the Berea which would be drawn into the central area. This would be accompanied by a change in the catchment area and actual recharge but this cannot be considered further here.

b. There would be a change in the position of the salt water/ground water boundary to the south and east of Durban.

5.2 Effect on salt water/ground water interface.

A simplified method of determining the position of the salt water/ground water interface is given by the Ghyben Herzberg effect (14). This method, which implies no flow at the salt water/ground water interface, gives the depth below sea level Z_i to a point on the interface as being:

$$z_i = \frac{\rho f}{\rho s - \rho f} = h_f$$

where

ρ_f = density of ground water

ρ_s = density of salt water

h_f = head of fresh water above sea level at the point on the interface

This equation is based on the assumption that the less dense ground water will float on the denser salt water if undisturbed.

In reality the interface is not static and according to Cooper (10) the salt water flows perpetually in a cycle from the floor of the sea into the zone of diffusion and back to the sea, and this flow tends to lessen the extent to which the salt water occupies the aquifer. Any reduction in ground water flow will cause a migration of the interface inland, possibly to an extent where the salt water will be drawn into the dewatering system and generally be close to the ground surface below the City. A number of wells in coastal cities throughout the world, including Long Island New York, Los Angeles California, and Oahu Honolulu, have salted up as a result of this affect (20).

5.2.1 Effect of an inland migration of the salt water/ ground water interface.

If a migration inland of the salt water/ ground water interface occurs, the structure of the soil which was previously in the ground water zone will be altered. It is known (2,35) that the clay minerals in such soils are a function of the mineral composition at the time of salinisation and the length of time the soil has been salinised. If recently salinised the structure of the minerals in the colloidal fraction will not be immediately affected and the commonly occuring clay minerals may be expected in the same proportion as that prior to salinisation (12), although the ion exchange mechanism may be vastly altered.

Sea water itself is a complex solution containing a large number of elements (ions, gases, organic matter) and among the chemical elements are chloride (55%), sodium (30%), sulphate (7%), magnesium (3,7%) and potassium (1,1%). Pickard (1964) noted that the range of surface water salinity in the open ocean is 33-37 g/l.

Whilst a detailed discussion on the effects of sea water on soil structure is outside the scope of this work, a brief review is worthwhile. References (2, 12, 35) should be consulted for further reading.

Basically the changes are brought about by reduction and base exchange. The reduction processes, mainly of a biochemical nature, influence the concentration of SO_4 in the ground water. While the ground water is flowing the soil acts as an ion exchanger and cations in the water reach equilibrium with the soil cations.

These changes have implications on the soil permeability since any process which will alter the pore size distribution in a soil will alter its permeability. The dispersion or peptisation of soil colloids and the opposing process of flocculation are of paramount importance in this respect.

Where sodium adsorption in the subsoil is increasing, swelling of the soil colloids decreases the permeability and migration of clay particles with concurrent blocking of fine pores has the same effect.

Apart from the changes in the soil structure the affect of the salts upon building structures themselves will have serious consequences and even construction techniques and materials may need reassessing.

By far the most serious effect will be the possible deterioration of concrete in foundations by sulphate attack. These affects are well known and documented in the many works on concrete technology. The sulphates in solution react with Portland cement to form insoluble calcium sulphate and calcium sulpho-aluminate. An increase in molecular volume takes place and this, together with crystallization of new compounds, causes expansion and disintegration of the concrete surface exposing new areas to further attack.

The concentration of sulphates in the ground water below the City of Durban at the present time (1973) is below the critical level of 30 parts SO_3 per 100,000 and no special precautions have been necessary. The volume of water which requires to be pumped to increase this and cause the above effect is equivalent to the 'safe yield' concept in ground water supply and varies with recharge and location of site. There are numerous methods (11,32) of estimating the safe yield but in general the results are unreliable and most depend as a safeguard upon continuous monitoring of ground water samples and levels. The safe yield is however always less than the natural recharge and in coastal areas it could be considered to reduce as the site approaches the salt water coastal aquifer, i.e. it decreases towards the south of the City centre and increases as the site is located further to the north west. The fact that a reduced volume of pumping is able to create inflow conditions as the well approaches the Durban

Bay is partly offset by the smaller area which will be affected by the inflowing salt water.

Salt water inflow over very short periods due to minimum recharge conditions is unlikely to be serious as the fresh ground water flow under normal recharge conditions will be sufficient to leach out any of the salts deposited despite the fact that this effect will take longer to complete than the deposition (12).

5.3 Effect on Settlements.

It is now well known that excessive pumping of ground water from beneath many of the world's cities, particularly at coastal areas, has led to large settlements of the cities concerned.

Perhaps the best known example is Mexico City (5) which has been well documented but similar effects, if less dramatic, have been reported by Legget (20) and Poland & Davis (27) at Tokyo, London and Los Angeles. A short description will indicate the extent of the problem.

Mexico City lies in the west central part of the Valley of Mexico, which is a closed basin at an average elevation above sea level of 2500m and is surrounded by high mountains. The settlements in Mexico City have been going on for several hundreds of years and are considered to have begun as the result of early drainage and flood prevention schemes by the occupants of the City. The settlements increased in the late 1700s due to more extensive drainage and at the current rate of pumping from some 3500 artesian wells the present settlement is of the order of 1mm/day. Some of the wells extend to a depth of 200m although most are in the 15-150m range. Since 1900 the City has settled 5m and in 1944 the rate of pumping was $7m^3$/sec. Between 1900 and 1957 settlements of up to 7,5m were recorded on some buildings.

The problems of settlements of this order are obvious. Sewers and pipes break, buildings and monuments tilt and crack, etc. The first floor of the Palace of Fine Arts, for example, is now the basement and even modern buildings cannot escape settlement.

Fortunately the geology of the Mexico area is vastly different to Durban and settlements of similar magnitude could not be repeated in the Durban area. In fact the vast settlements in Mexico are largely due to the unusual nature of the soil. The soil consists of layered gravel, sand, silt and clay, and borings have gone to 600m without hitting bedrock. The clay (which is interspersed with lenses and layers of sand) ranges in depth from 10-180m and is organic, chiefly of montmorillonite. It has a very low unit weight, a high moisture content and low permeability. 200-500% moisture by dry weight has been recorded from undisturbed samples, with $k = 10^{-7}$ cm/sec, a void ratio e of from 2-15, and the submerged weight is 38 lb/ft^3.

In Japan at Tokyo, Nagoya and Osaka settlement has also occurred. At Osaka it is attributed to the consolidation of alluvial clay 30m thick. Between 1885 and 1928 settlement occurred at the rate of 6-13mm/year. Since then pumping has increased and settlements up to 3m have been recorded. During the war when pumping stopped the settlement also ceased.

At Long Beach, California, up to 8m of settlement has occurred through dewatering, and although many occupants would not have noticed even London has settled due to abstraction of the ground water from the London clay. Precise levelling since 1865 indicates 21,4mm settlement near Hyde Park accompanied by a drop in water level from 9m above sea level in 1820 to as low as 90m below sea level to date.

The range of this problem can thus be seen and Durban, with its mainly sandy and relatively shallow (35m) soil profile, could be expected to lie somewhere in between the extremes given above and at the lower end of the scale. To verify this the settlement has been estimated below for a typical soil profile but due to the intermittent occurrence and variability of the clay layers and in particular the difficulty of obtaining representative samples of the sands for testing in the consolidometer, no attempt at a locally precise settlement calculation has been made. The order of magnitude based on the 'typical' soil profile (see boreholes in Appendix A) is as realistic as would be any more refined calculation based on the imperfect soil characteristics available. Once again each site must be studied and analysed separately if, in the light of the above, a problem is believed to exist.

Compression of the soil is dependent on a change in effective stress $\bar{\sigma}$ given by

$$\bar{\sigma} = \sigma - \mu$$

where

σ = total stress on the soil

μ = pore water pressure

Assuming that the total stress, which comprises overburden of both soil and water plus any applied stress at the surface, remains constant, then any reduction in pore water pressure causes an increase in effective stress with associated compressions. Assuming that any reduction in water level occurs over a large area the effective stress change is one dimensional and vertical compression only need be considered.

The settlement of a typical profile may be calculated from Dutch Probe results using the relationship

$$\rho = \frac{H_c}{c} \log_e \frac{P_o - \Delta p}{P_o}$$

where $c = 1,5 \frac{Ckd}{P_o}$

and Ckd = point resistance lb/sq.in. at depth d_c

P_o = effective overburden pressure at depth d_c

Δp = change in effective overburden pressure P_o

H_c = thickness of layer.

D.P.1 in Appendix A was taken at a position close to BH.3 and was used to derive the settlements in Fig.5.1 A total density of 120 lb/ft.^{3} (18,9 KN/m^2) was taken and a saturated density of 130 lb/ft.^{3} (20,4 KN/m^2). The water table was 9ft. (2,7m) below ground level.

The value of the point resistance below 51ft. (15,5m) was assumed constant at 2,0 Kip/sq.in. (13,8N/mm^2) in view of the almost constant SPT values below that depth. The settlement was calculated over a total depth of 90ft. (27,4m).

The Dutch Cone point resistance was divided into layers and average Ckd values taken for each layer. The settlements have been calculated and are plotted against drawdown of the water table in Fig.5.1. The magnitude of settlement is not large and if uniform settlements could be assured no serious problem would arise. Unfortunately this situation is most unlikely and steep drawdowns over small radii of influence, as obtained in section 4.2, may be expected to give differential settlements of up to 1 vertical to 200 horizontal. According to Skempton and MacDonald (29)

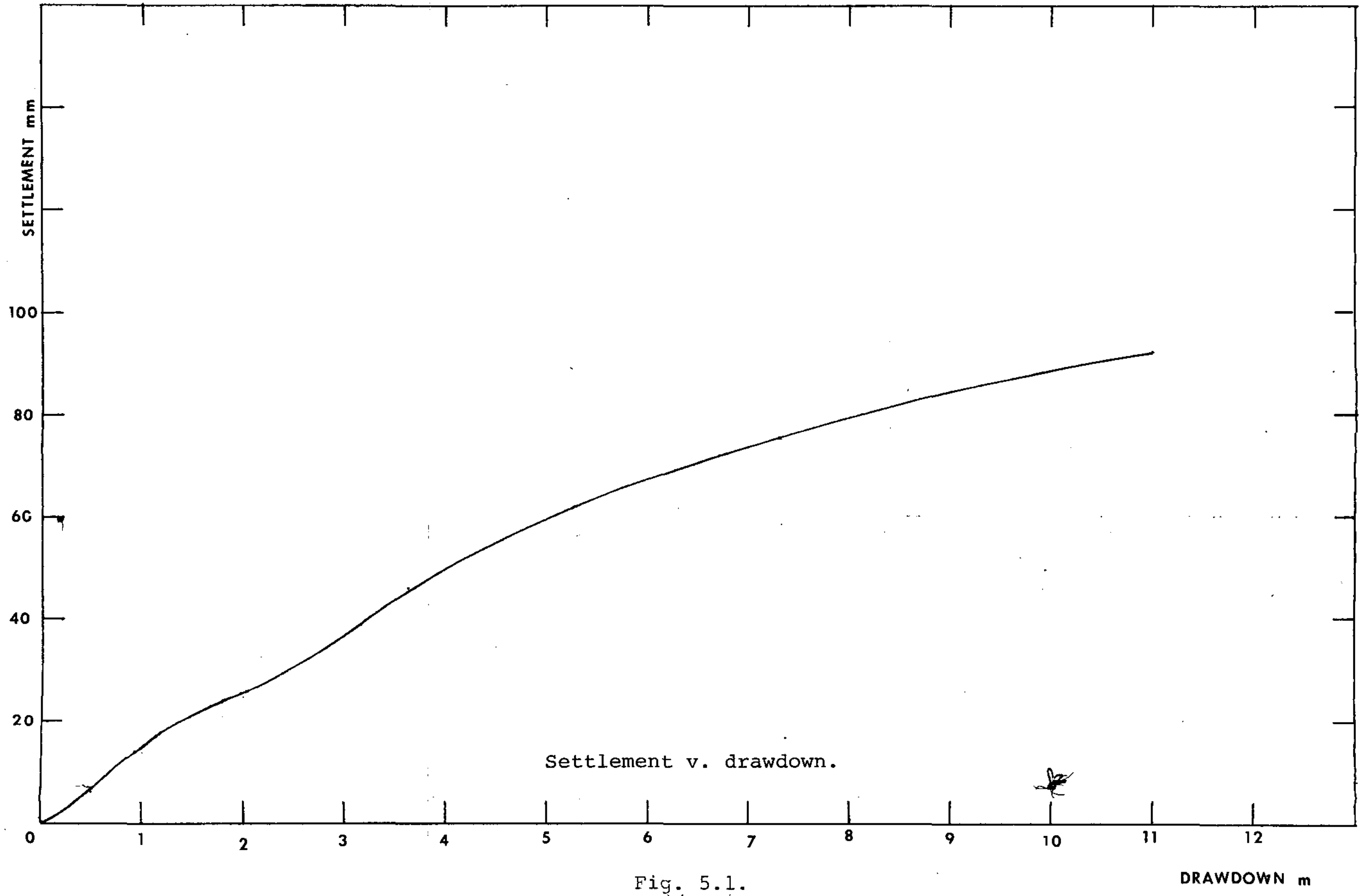

SETTLEMENT mm
100
80
60
40
20
0
1
2
3
4
5
6
7
8
9
10
11
12
DRAWDOWN m
Settlement v. drawdown.

Fig. 5.1.

this is likely to cause cracking of brickwork but is unlikely to cause severe structural damage to framed buildings. However, the settlements are of sufficient magnitude to warrant individual checks on all adjacent property before major dewatering is undertaken.

No major problems in the Durban area as a result of either differential or total settlements are expected.

6.0 Conclusions.

Using the estimated average coefficient of permeability for the Durban area it has been shown that the extraction of ground water from excavations in the City in quantities approaching the estimated daily recharge is a possibility.

The consequences of this overpumping are a change in the flow patterns and recharge to the City from the inland area to the east and north of the City, and more importantly from the Indian Ocean and the Durban Bay from which there would be an inland migration of salt water. The effect of the inflowing salt water on the soil structure and on building foundations and services will depend on the salt concentrations present. Although these concentrations cannot be accurately predicted on present information the consequences of a deterioration of building foundations are such that pumping should be carefully controlled until such information becomes available. As a measure of control and to allow for variations in recharge, quantities in excess of 1000 m^3/day should not be permitted to be pumped from sites in the centre of the City unless definite means are taken to ensure that long term inflow conditions would not occur. This control could take the form of a grid of piezometers (of which the existing experimental piezometers could form a part) to monitor the ground water levels together with regular sampling of the ground water from these positions. In addition, basements which incorporate permanent dewatering systems as a part of the design should not be permitted, thus avoiding cumulative extractions and permitting an eventual return to the original flow conditions.

The amount of infiltration from inland areas to the east and north of the City is critical in determining the amount of dewatering which may be permitted, and care should be taken that building and road coverage where surface water drains to main drainage does not increase to an extent where infiltration is significantly reduced.

The lowering of the ground water table will not present major settlement problems for the City (i.e. problems similar to Mexico City). Nevertheless, settlements of the order of 100mm for 12m drawdown could occur and the effect of differential settlements will need to be considered locally.

Further research on the following is essential if accurate values of the effect of water table lowering are to be predicted.

a. Evapotranspiration under local conditions.
b. The catchment area and the effect of flow path variations on recharge.
c. The position and effect of the former course of

the Umgeni River.

d. Further field pumping tests to obtain greater accuracy in average values of the coefficient of permeability.

e. The extent of the salt water/ground water interface migration and the salt concentrations for various pumping conditions.

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A P P E N D I X   A

Borehole Log & Penetration Test Data

Borehole No.
B.H. 1
Date Drilled. May 1967

R.L.+ 24′

Dark greyish brown medium SAND.

Grey and yellowish brown mottled sandy CLAY.

Yellowish brown and reddish brown mottled fine SAND.

Yellowish brown and reddish brown clayey fine SAND.

Yellowish brown and reddish brown clayey medium SAND.

Grey and yellowish brown CLAY.

Light grey fine SAND.

Grey fine SAND.

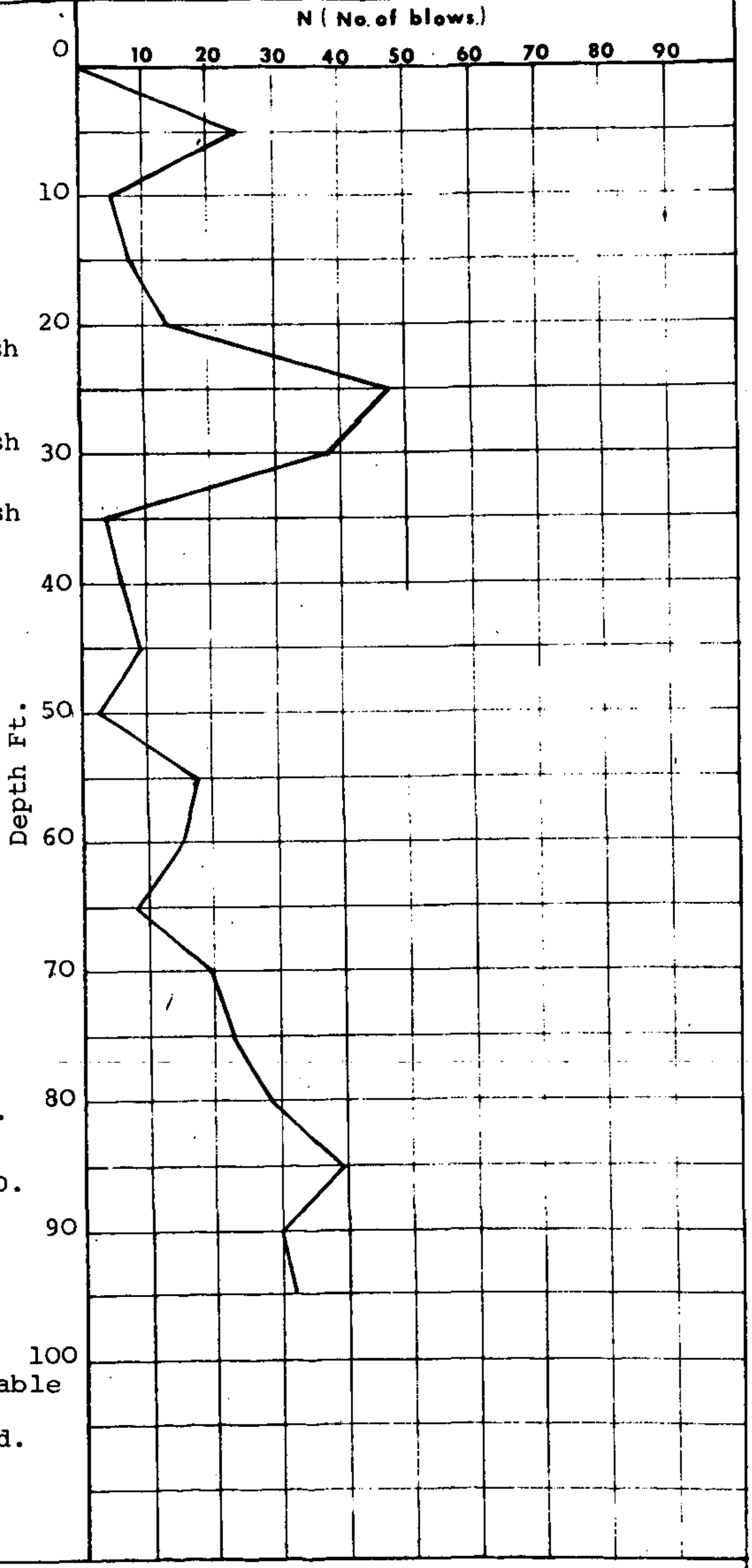
Light grey and yellowish brown mottled fine SAND.

Yellowish brown fine SAND.

Dark grey clayey fine SAND.

Dark grey CLAY with shell fragments.

Light yellowish brown friable calcareous sandstone and yellowish brown dense sand. Weathered Cretaceous.



Borehole Log & Penetration Test Data

Borehole No. B.H. 2

Date Drilled. Sept 1970

R.L.+ 23′

Rubble

Brown SAND.

Yellow-grey soft sandy CLAY.

Yellow-grey clayey SAND.

Slightly clayey SAND.

Light grey fine CLAY-SAND.

Clayey SAND.

Grey slightly clayey SAND.

Fine silty SAND.

Medium to coarse SAND.

Very fine silty SAND.

Medium to coarse SAND.

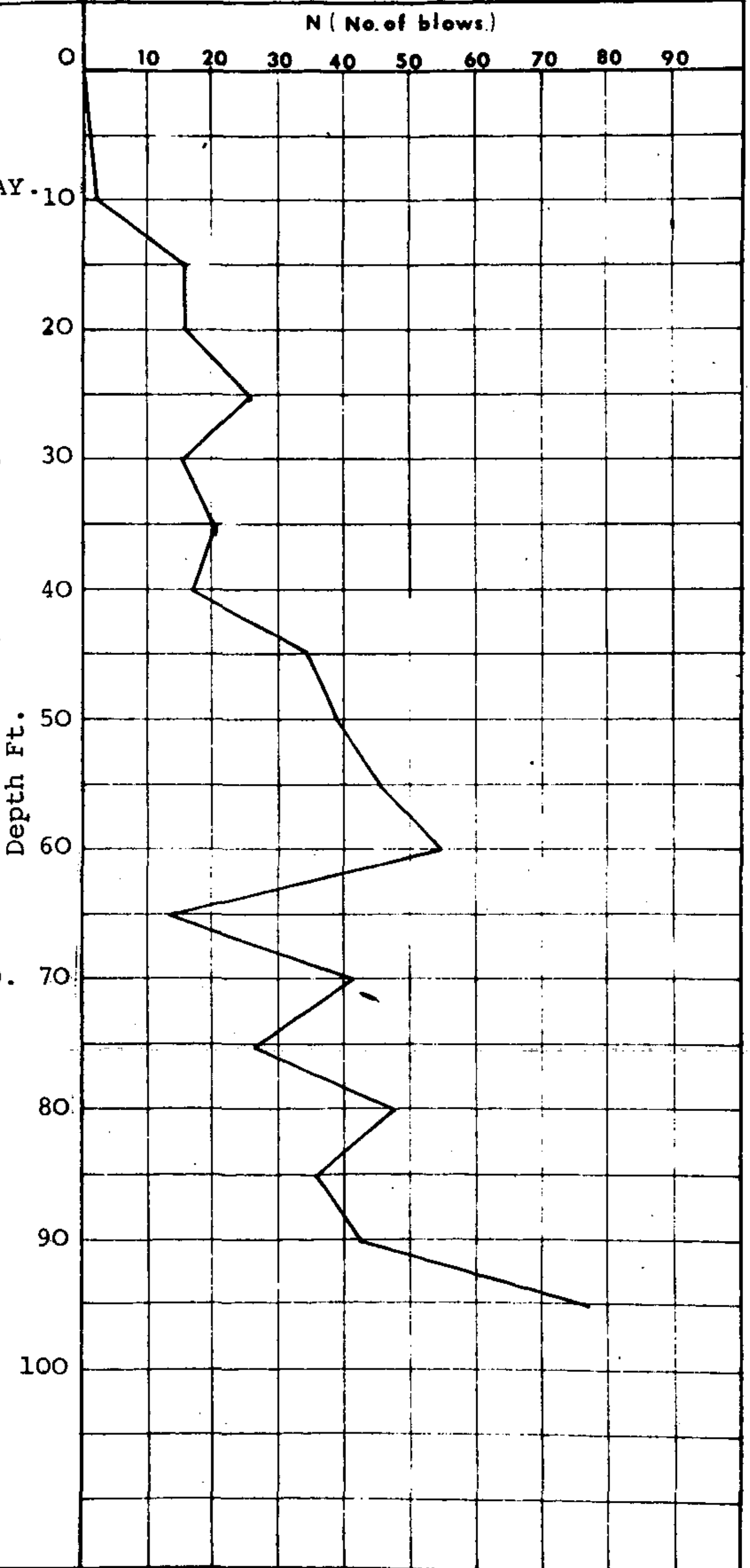
Soft SAND-CLAY.

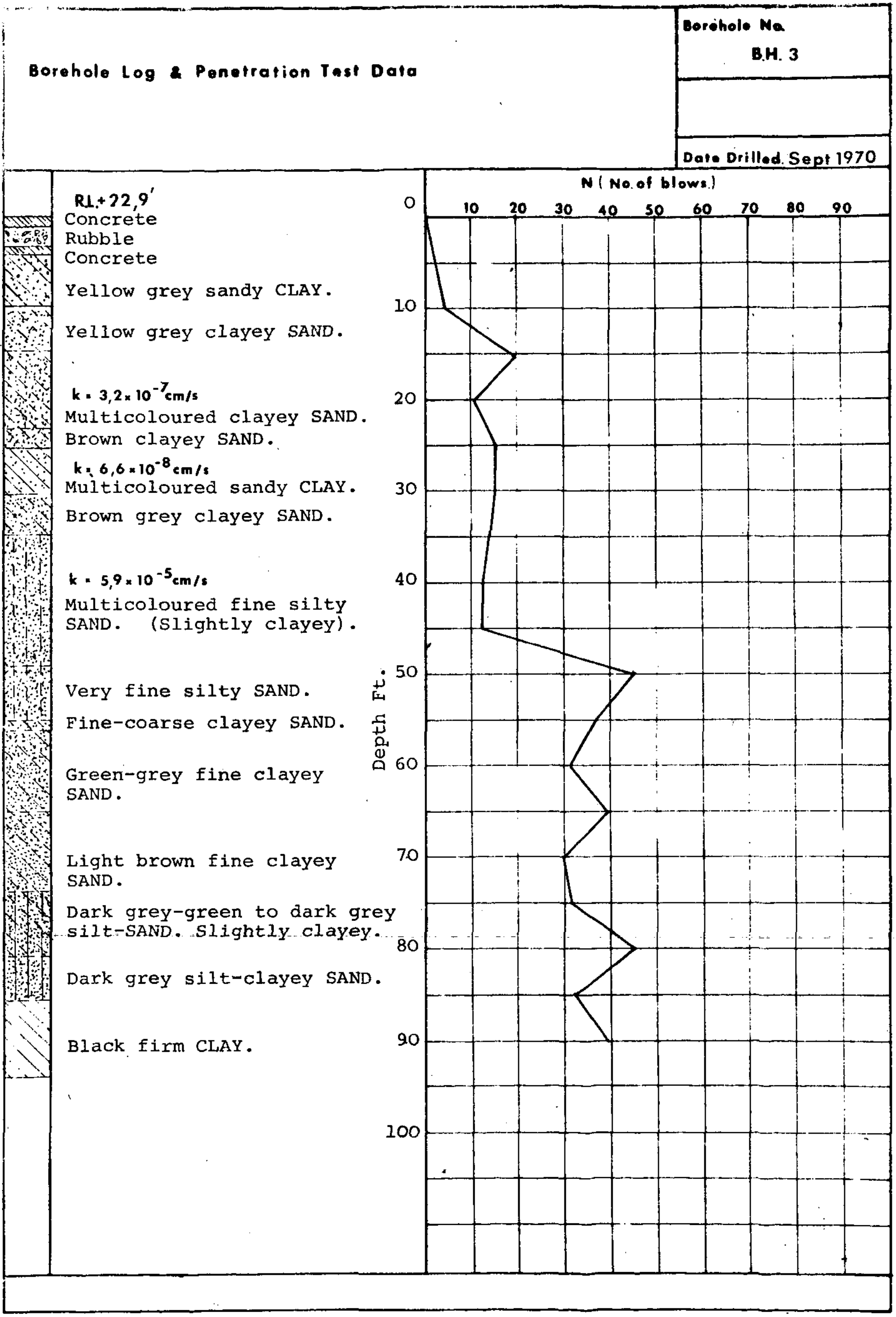
Slightly clayey silty SAND.

Black silty CLAY.

Black silty CLAY with sea shells.

Very fine silt-SAND slightly clayey.




Borehole Log & Penetration Test Data
Borehole No.
B.H. 3
Date Drilled. Sept 1970
N (No. of blows.)
10 20 30 40 50 60 70 80 90
Depth Ft.
0 10 20 30 40 50 60 70 80 90 100
RL.+22,9′
Concrete
Rubble
Concrete
Yellow grey sandy CLAY.
Yellow grey clayey SAND.
k = 3,2 x 10⁻⁷ cm/s
Multicoloured clayey SAND.
Brown clayey SAND.
k = 6,6 x 10⁻⁸ cm/s
Multicoloured sandy CLAY.
Brown grey clayey SAND.
k = 5,9 x 10⁻⁵ cm/s
Multicoloured fine silty SAND. (Slightly clayey).
Very fine silty SAND.
Fine-coarse clayey SAND.
Green-grey fine clayey SAND.
Light brown fine clayey SAND.
Dark grey-green to dark grey silt-SAND. Slightly clayey.
Dark grey silt-clayey SAND.
Black firm CLAY.

Borehole Log & Penetration Test Data

Borehole No. B.H. 4

Date Drilled. July 1970

R.L.+19′

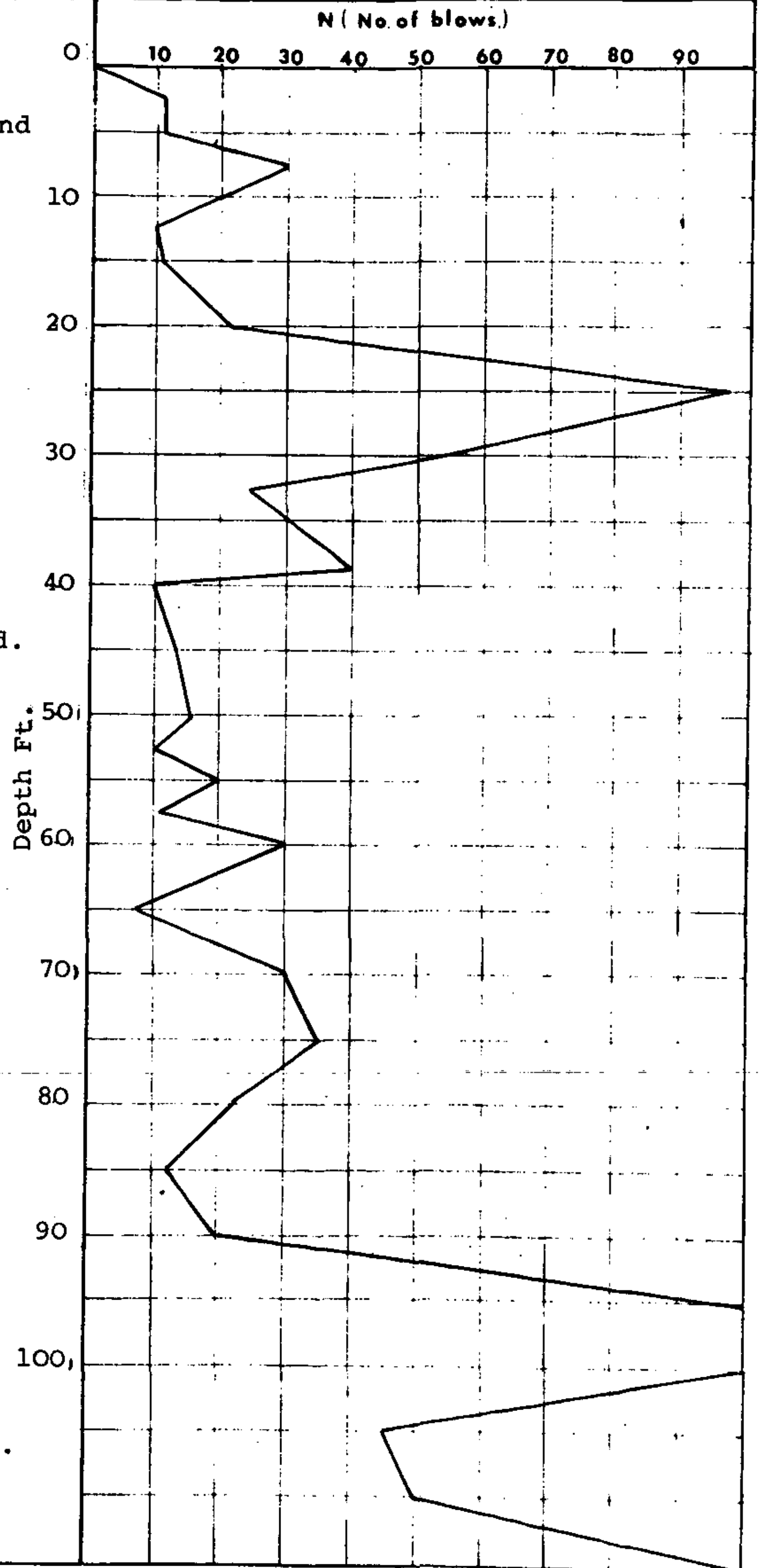
Dark brown, grey, olive and yellow, medium dense and dense fine- and coarse-grained sand, with olive, grey, soft silt and clay layers, Harbour Beds.

Yellowish orange, very dense fine sand.

Grey, white, yellow and olive, medium dense, fine sand with layers of clayey silt and silty sand.

Dark grey, firm, clayey silt.

Dark grey and brown, very dense, silty fine sand with thin clay layers containing rounded pebbles.



Borehole Log & Penetration Test Data

Borehole No.	B.H. 5
Date Drilled.	Nov 1971

R.L.+19,5'

Light brown loose intact medium and fine SAND. Harbour Beds.

$k = 7{,}1 \times 10^{-10}$ cm/s
Olive grey very stiff intact CLAY. Harbour Beds.

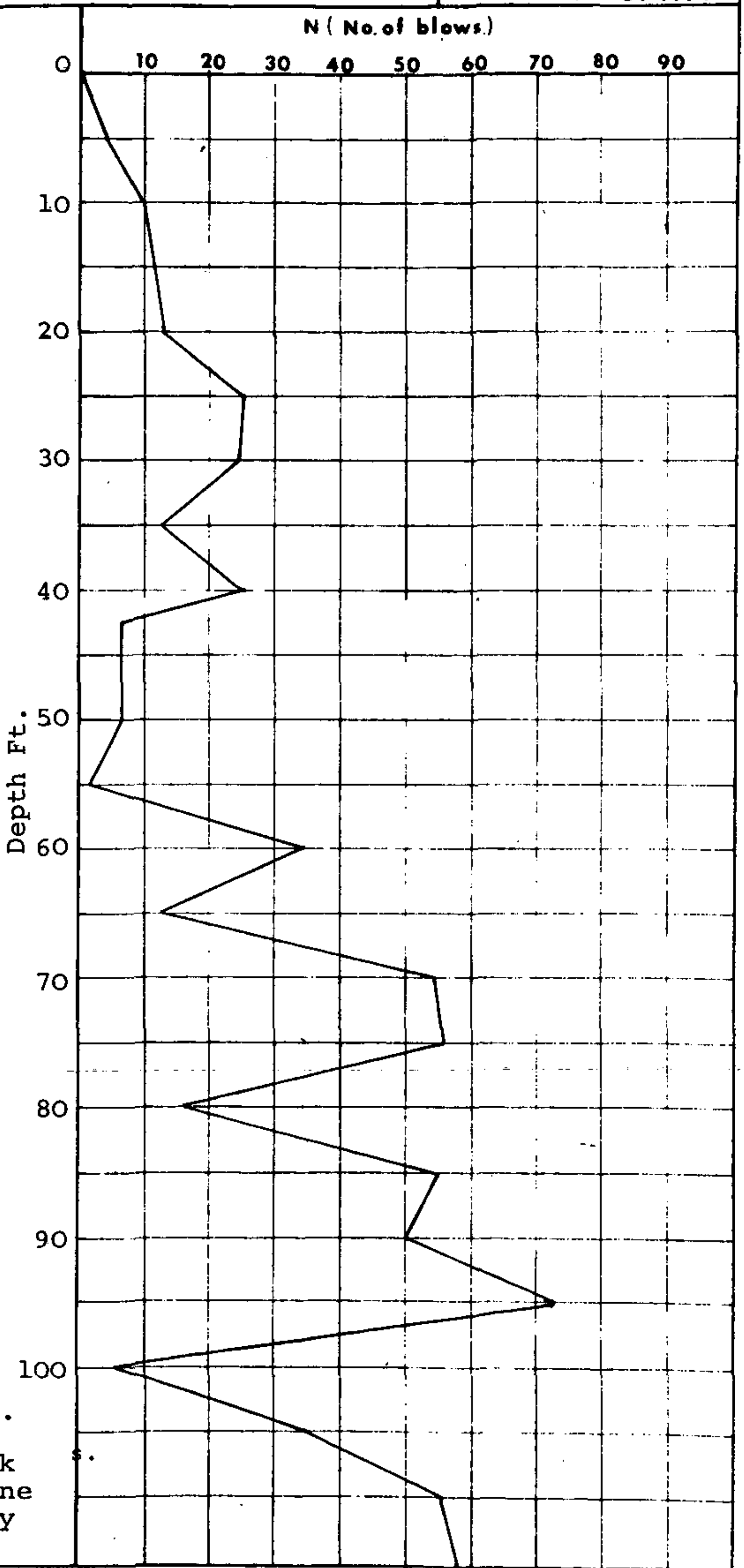
Light grey brown loose intact slightly silty fine SAND. Harbour Beds.

As above but dense.

Olive grey firm intact CLAY. Harbour Beds.

Dark grey dense intact slightly silty fine SAND. Harbour Beds.

Light greyish white loose coarse SAND. Harbour Beds.
Black firm intact CLAY. Harbour Beds. Light to dark grey dense intact silty fine SAND with occasional shelly bands. Harbour Beds.



Borehole Log & Penetration Test Data

Borehole No. B.H. 6

Date Drilled. Nov 1971

R.L. + 19,5′

Light brown medium dense intact medium and fine SAND. Harbour Beds.

As above but loose.

Light brown mottled yellowish orange very dense intact fine SAND. Harbour Beds.

$k = 1{,}4 \times 10^{-7}$ cm/s

Light grey brown loose to medium dense clayey SAND. Harbour Beds.

Dark reddish brown dense intact fine SAND. Harbour Beds.

Grey dense intact silty fine SAND. Harbour Beds.

Black firm to stiff intact CLAY. Harbour Beds.

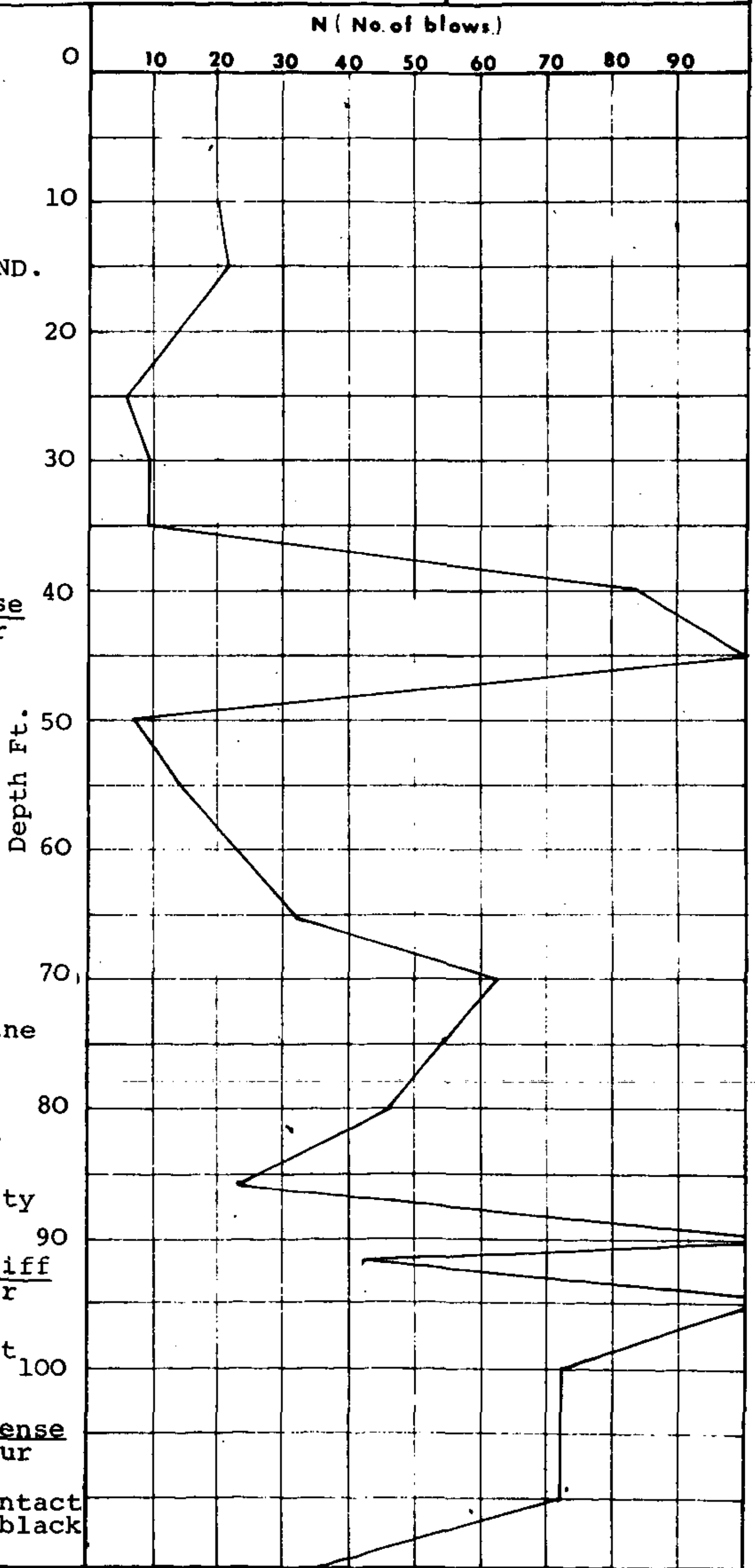
Dark grey dense intact silty fine SAND. Harbour Beds.

Dark grey to black very stiff intact sandy CLAY. Harbour Beds.

Dark grey very dense intact clayey silty SAND. Harbour Beds.

Light greyish white very dense intact coarse SAND. Harbour Beds.

Dark grey to black dense intact fine SAND with patches of black CLAY. Harbour Beds.



Borehole Log & Penetration Test Data

Borehole No. B.H. 7

Date Drilled Aug 1972

R.L. + 11,4'

D/reddish brown and d/yellowish orange medium dense silty fine and medium SAND.

D/yellowish orange mottled l/grey and l/brown medium dense slightly clayey fine and medium SAND.

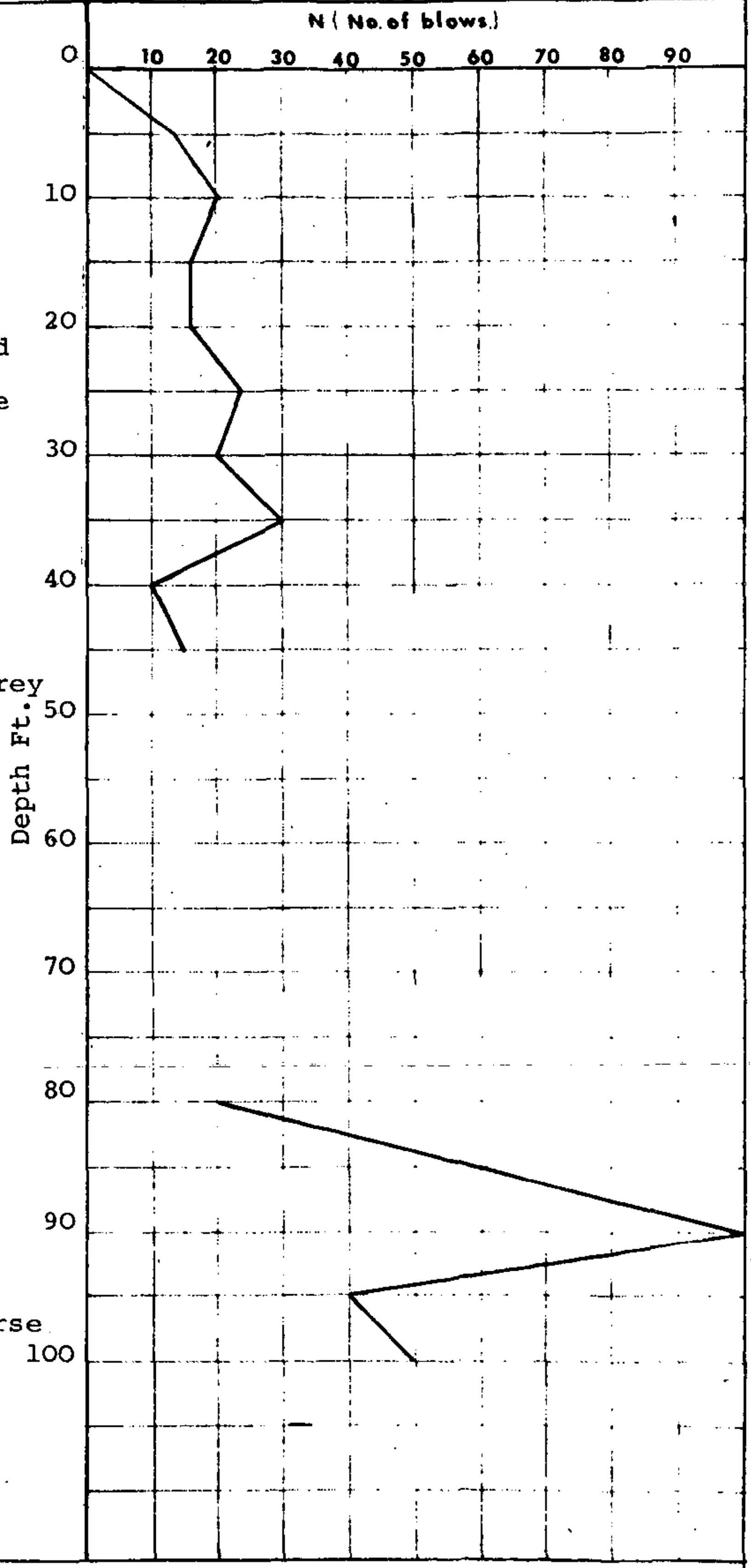
k = 1,5 × 10^{-6} cm/s

L/olive brown mottled l/grey and l/brown loose and medium clayey fine and medium SAND.

D/grey v/stiff shattered silty CLAY.

D/greyish brown and black v/dense silty fine SAND, becoming sandy silt with depth.

L/reddish brown dense coarse and medium SAND.



Borehole Log & Penetration Test Data

Borehole No.
B.H. 8

Date Drilled Aug 1972

R.L.+11,9′

D/orange brown slightly clayey fine and medium SAND.

L/brown and d/brown slightly clayey medium SAND.

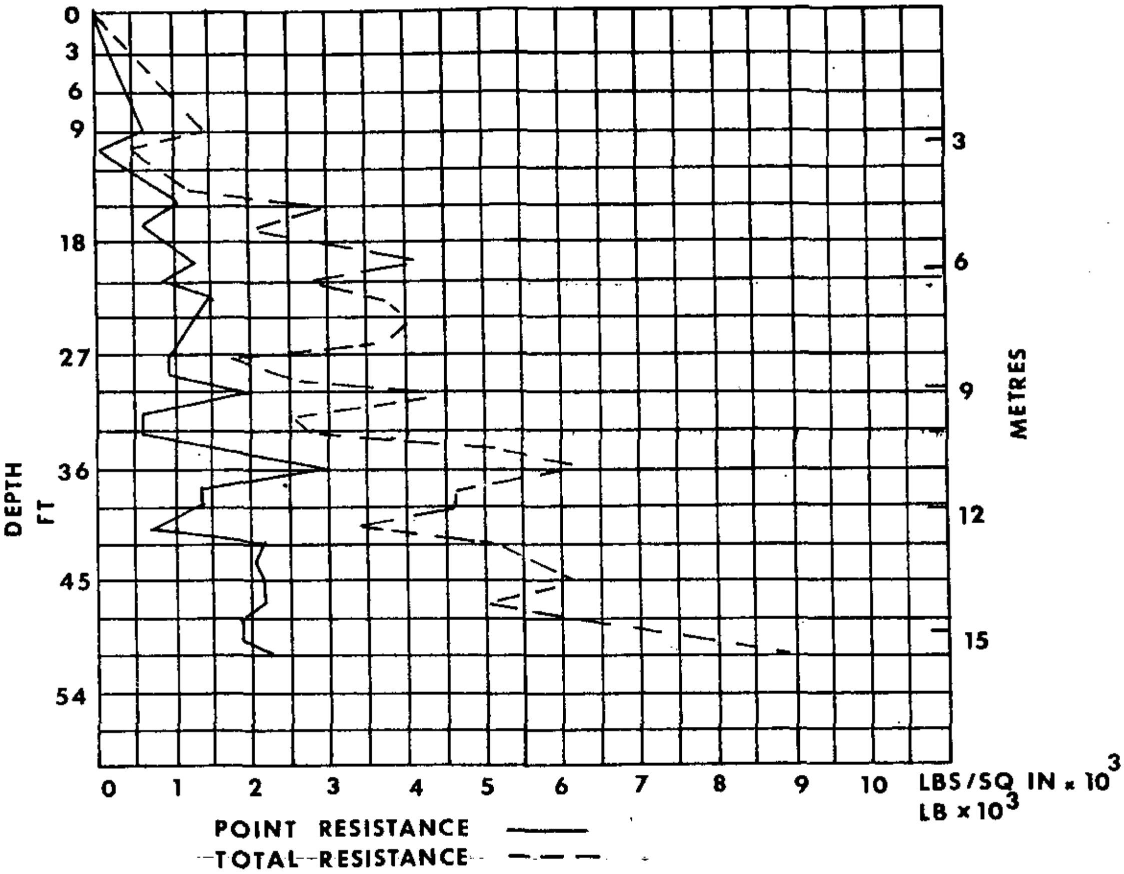
L/grey mottled l/brown and l/yellowish brown shattered silty CLAY.

L/grey clayey fine and medium SAND.

D/olive brown clayey fine and medium SAND.
$k = 5,4 \times 10^{-8}$ cm/s.

Depth Ft.: 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100

N (No. of blows.): 10, 20, 30, 40, 50, 60, 70, 80, 90


0
3
6
9
18
27
36
45
54
DEPTH
FT
3
6
9
12
15
METRES
0 1 2 3 4 5 6 7 8 9 10
LBS/SQ IN x 10^3
LB x 10^3
POINT RESISTANCE
TOTAL RESISTANCE

DP1