SOME NOMOGRAPHS FOR
THE CALCULATION OF DRAIN SPACINGS

Bulletin 8

SOME NOMOGRAPHS FOR THE CALCULATION OF DRAIN SPACINGS

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1. INTRODUCTION

The purpose of drainage in *humid* and *semi-humid areas* is to create an aerated root zone and a surface soil which is dry enough to enable farm implements and machinery to be used whenever they are needed. Surplus water (usually in the form of rain-water) must therefore be removed and the water table maintained at the level which is most appropriate for the particular soil, crop and climate.

In most *irrigated areas* drainage is required to control the rise in the water table due to conveyance losses and over-irrigation, thus preventing waterlogging and salinization.

When *saline land* is reclaimed, good drainage facilities are a prerequisite for leaching excess salts out of the root zone, desalinizing the ground water and preventing resalinization.

In recent years the importance of good drainage has come to be increasingly realised. Moreover areas which could not be utilised owing to an excess or lack of water are now being brought into cultivation by means of drainage and/or irrigation works. Consequently there has been a very considerable increase in the need of aids to the design of a drainage system.

In this design a very important place is occupied by the calculation of the proper spacing of parallel relief drains. For this purpose numerous drainage formulas have been developed over the years.

In order to use these formulas and corresponding nomographs two entirely different set of data are required, viz. the *drainage criteria* and the *physical conditions of the hydrological soil profile*.

By 'drainage criteria' or 'drainage requirements' is here meant the minimum permissible depth of the ground water and the corresponding projected discharge (i.e. the amount of water to be removed). In the case of formulas based on steady-state flow conditions the physical soil data required are the hydraulic conductivity and the depth to an impermeable layer, whereas in the case of non-steady flow formulas it is also necessary to know the drainable pore space.

In this bulletin it is assumed that the data required for using the formulas (the drainage criteria and hydrological soil characteristics) are already known. As regards the general theoretical bases for the calculation of ground-water flow, etc., reference may be made to the literature on this subject. Here an exception is only made in respect of that portion of theoretical knowledge which is considered desirable for the proper use of the formulas and nomographs.

Since the nomographs presented here have greatly simplified the use of the various drainage formulas, it might possibly be inferred from this that it is also a very simple matter to design an efficient drainage system. But this is not altogether correct.

The importance of a good drainage formula for planning the proper drain spacing is often overrated. The chain is only as strong as its weakest link. The weak link in the calculation of drain spacing is not the formula itself but the data substituted in it, viz. the drainage criteria and hydrological soil data. Since the hydrological soil data in particular may vary extensively within a short distance, their accuracy is very often in doubt. It should, therefore, be realised that in practice the use of drainage formulas can never result in more than what may be termed a 'calculated estimate', viz. a drain spacing which very probably will be approximately correct. Unlike the sphere in which the chemist or physicist works, it is true of many agro-hydrological calculations: from science to fiction is but one step.

But on the other hand the importance of the formula should not be underrated. It can not only be employed for calculations, but in a summary form it can afford a good idea of the various factors involved and their relative importance under different conditions.

Although many different types of nomographs are possible for the formulas presented here, the aim has been to elaborate a particular type of nomograph which would enable workers to ascertain, as readily as possible, the relative importance of the various factors or the extent to which the final result is influenced by the greater or lesser accuracy with which the magnitude of a particular factor is determined. Examples of this will be given when discussing the various nomographs.

In the notation employed for the formulas, the aim has been to take into account both international and ready use. Capitals are used for the drain spacing to be finally calculated L and for the soil data required, viz. K (permeability), D (thickness of a layer) and V (drainable pore space). Lower-case letters are used for other data (h, q, r, u). Use of the same capital and lower case letter (H, h) is avoided as far as possible; this use is clear in writing but causes difficulties in a spoken discussion. Except for π, Greek letters are avoided in order to facilitate the typing of formulas in reports, etc.

Finally it should be pointed out that the various nomographs were compiled on the assumption that persons responsible for drainage calculations are familiar with the use of a slide-rule.

2. GENERAL REMARKS

2.1. STEADY AND NON-STEADY FLOW CONDITIONS

Most drain-spacing formulas, including those of HOOGHOUDT and ERNST as used in this publication, were developed on the basis of steady-state flow conditions, e.g. continuous steady rainfall, discharged continuously and steadily by the drains, in other words for a state of equilibrium between supply and discharge.
This situation will rarely occur in practice. In practically every case we shall have non-steady-state conditions, i.e. a rising and falling water table. Despite this fact the use of a steady-state formula is in many cases entirely justified.

In order to calculate the required drain spacing in a given hydrological situation the required intensity of the drainage system should be known. This drainage intensity is given in quantitative terms by the *drainage design criteria*. These vary for steady and non-steady formulas.
For the non-steady formula (as employed in the present publication) the drainage intensity is determined by the required fall in the water table over a given number of days, starting from a given initial state (fall from h_0 to h_t over a time t).
For the steady-state formula, the drainage design criteria for a given drain depth are the maximum permissible height of the ground-water midway between the drains and the corresponding projected discharge, viz. a given combination of h and q. The design discharge q is determined by the mean rainfall distribution; the available hydraulic head h being determined by the depth of the drains and the minimum permissible depth of the ground water. The latter is an agronomic criterion (in the Netherlands it is 0.5 m for arable land and 0.3 m for grassland).

In this connection it should also be noted that different combinations of q and h can produce the same drain spacing. Thus for a drain depth of 1 m the combination $q =$ 7 mm/day and $h = 0.5$ m gives practically the same drain spacing or the same intensity of the drain system as the combination $q = 10$ mm/day and $h = 0.7$ m.
The steady-state formulas can be used for conditions in which, as in Europe, rainfall is usually prolonged and of slight intensity. Experience will then tell whether the calculated intensity of the drainage system is in fact sufficient. This can be inferred from the depth of the ground water during the year and its consequences for agriculture.

But in irrigated land, or in areas of short and intense rainfall, there is a very great difference in the intensity and duration of the supply and discharge of water. Hence in order to calculate drain spacing under such conditions it is desirable to use a drain spacing formula which is based on non-steady or transient flow conditions (DUMM 1954, KRAIJENHOFF 1958, MAASLAND 1956, 1959).
It is simpler to calculate drain spacings by means of the steady-state formulas than by the non-steady state formulas. Therefore in many cases in which the non-steady conditions cannot be properly defined, or the hydrological constants are only approximately known, the use of steady-state formulas is justified.

2.2. DRAINAGE FORMULAS BASED ON STEADY-STATE CONDITIONS; VERTICAL, HORIZONTAL AND RADIAL FLOW COMPONENTS

A very good approximation of the ground-water flow to a parallel drainage system can be obtained by analysing the flow into a *vertical*, a *horizontal* and a *radial component* with corresponding resistances (HOOGHOUDT 1940, ERNST 1954) (Fig. 1). By radial resistance is here meant the flow restriction due to the convergence of the flow lines as they approach the drains.

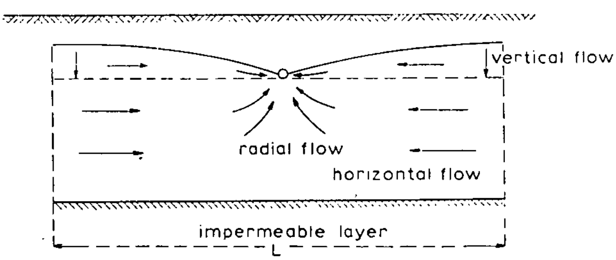


Fig. 1. Vertical, horizontal and radial flow components

The vertical resistance is usually very small, so that the required drain spacing is mainly determined by the horizontal and radial resistances that occur.
The extent to which the horizontal or radial resistance predominates, depends mainly on the position of the impermeable layer (the 'barrier layer') with respect to the drains. We can distinguish three cases, viz.:

a. When the *drains are situated on the impermeable layer or the latter is a short distance below the drains*, the horizontal resistance predominates and the radial resistance is so small that it can be ignored.
b. When the *impermeable layer is at a great depth* ($> {}^1/_4$ drain spacing), the radial resistance predominates to such an extent that the horizontal resistance may be ignored.
   In both cases the corresponding drainage formulas and nomographs are very simple.
c. When *the depth of the impermeable layer below the drains is less than ${}^1/_4$ of the drain spacing*, the situation becomes more difficult. In this case the radial resistance is too great to be ignored. If only the horizontal resistance was included the calculated drain spacing would be too large, as, for example, with the DONNAN formula commonly used in the U.S.A. (DONNAN 1946, ROE and AYRES 1954, ISRAELSEN 1962, S.C.S. Handbook 1959, etc.).

It is to the credit of HOOGHOUDT (1940) that he found a solution (summarised in a number of tables) for the most difficult and frequent case *c* in which the impermeable layer is at some depth below the drains ($< {}^1/_4$ drain spacing). It was afterwards found that HOOGHOUDT's solution showed good agreement with the results of other investigations (hodograph method, relaxation method, model tests, etc.). Moreover HOOGHOUDT's solution could be used in practice. But although the formula is very simple, the calculations with this formula are rather cumbersome since, as will be illustrated below, it necessitates a process of trial and error.

2.3. NOMOGRAPHS

It has been found extremely difficult to draw a good nomograph for HOOGHOUDT's formula. A local solution has been developed in the Netherlands (VAN DER MOLEN, Grontmij Ltd. and VAN SOMEREN, Government Service for Land and Water Use) by taking a fixed discharge (q = 7 mm a day) and a fixed drain radius (r = 4 cm) and then compiling graphs for each of the most frequently occurring hydraulic heads (h = 0.2, 0.3 m, etc.).

A more general solution (all q-values) for tile drains ($r = 0.04$ m) in a homogeneous soil was proposed by ERNST and BOUMANS (VISSER 1954, LUTHKIN 1957, pp. 93 and 389). An attractive graphical solution of drain spacing formulas for homogeneous soil with an impermeable layer at a finite depth below the drains, which can be used for various r-values, is given in a recent publication by S. TOKSOZ and DON KIRKHAM (1961).

Compared with the above-mentioned graphs, those published in the present bulletin are more widely applicable, easier to use, and are suitable for a rapid determination of the relative effect of the various factors determining drain spacing.
The nomographic solution of drain spacing for tiles (Graph I) is based on HOOGHOUDT's formula. The other nomographs II, III and IV for steady-state conditions are based on ERNST's formulas, which have fewer limitations and are more suitable for a general graphical solution. For the non-steady state conditions (irrigation projects) use has been made of the GLOVER/DUMM formulas (Graph V).

3. NOMOGRAPHS BASED ON HOOGHOUDT'S FORMULA

HOMOGENEOUS SOIL BELOW THE DRAINS. IMPERMEABLE LAYER AT ANY DEPTH

3.1. EXPLANATION OF THE FORMULA

HOOGHOUDT's formula for computing the spacing between drains for a steady rainfall rate and with an impermeable layer at intermediate depth is as follows (see Fig. 2).

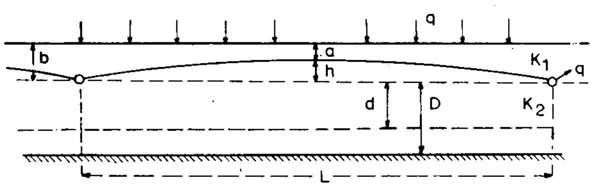


Fig. 2. Hooghoudt's formula

$$L^2 = \frac{8K_2 d\,h}{q} + \frac{4K_1 h^2}{q}$$

where

L = drain spacing in metres

q = drain discharge expressed in metres a day (m/day)

K_2 = hydraulic conductivity of the layer *below* the drains in m/day

K_1 = hydraulic conductivity of the layer *above* the drains in m/day

h = the height of the water table above the drain level midway between drains in metres

d = thickness of the 'equivalent layer', viz. a value depending on the drain spacing L, the drain radius r and the depth D of the impermeable layer below the bottom of the drain in metres.

The following remarks may be useful in connection with the formula:

- The first part of the formula relates to the flow *below* the drains, the second part to the flow *above* the drains.
- The formula is based on the presence of two layers of different permeability (K_2, K_1), the drains occupying the interface between these layers (Fig. 3b). For a homogeneous soil, $K_2 = K_1$ (Fig. 3a). If the drains are located below the interface (Fig. 3c) it is still possible to use HOOGHOUDT's formula, but without further calculations it is not possible to say what degree of accuracy will be attained.

  When the drains are above the interface (Fig. 3d) the HOOGHOUDT formula cannot be used. For this case ERNST's solution is given in Section 4.3.3., pag. 29.

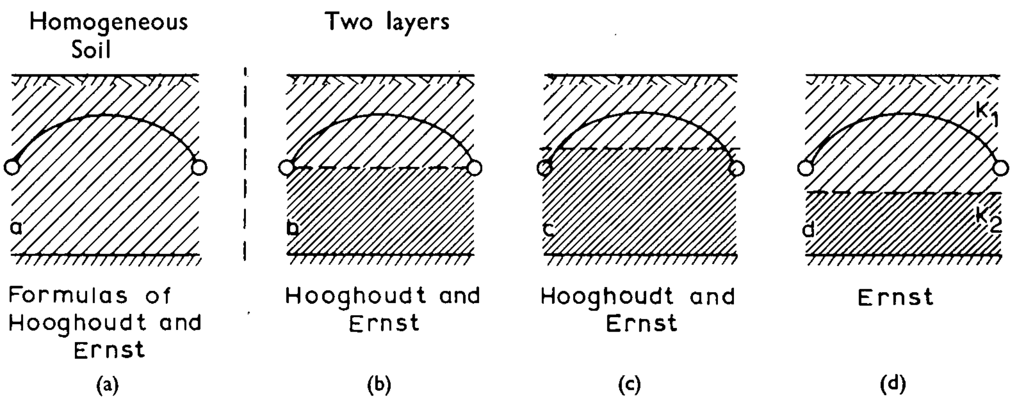


Fig. 3. Location of drains with respect to the interface of two layers

- The drainage coefficient q is here expressed in m/day, which is the same as m^3 *per m^2 area drained* (q = 0.005 m/day = 0.005 m^3/m^2). When the drain spacing is 40 m the *discharge per metre of tile line* is $qL = 0.005 \times 40 = 0.2$ m^3, and when this drain is 100 m long the *discharge of the tile line* will be $0.2 \times 100 = 20$ m^3 per day, or 20,000/86,400 = 0.23 litres per sec., and in this case the *discharge per ha* will be $0.23 \times 10000/(40 \times 100) = 0.58$ *lit/sec/ha*.

- The value of h, the available hydraulic head, can be calculated from the minimum permissible depth of the ground water (a in Fig. 2) and the depth of the drains b. The latter is a question of economics (deep drains permit wider spacing), the position of suitable soil layers, the level of available outlets, the salt content of the ground water, etc.
- If the impermeable layer is located at a very great depth, D can be taken as $^1/_4\, L$, since an impermeable layer at a depth $> {}^1/_4\, L$ has very little effect on drain spacing (HOOGHOUDT 1940, ERNST 1954). There are also many cases in which a depth of $> {}^1/_8\, L$ has little effect, but this should be checked by means of the nomograph.
- The value d, termed the 'equivalent layer', is introduced by HOOGHOUDT into his formula so as to take into account the radial flow in the vicinity of the drains. It is often found difficult to understand properly the meaning of this term which is therefore explained more fully further on.
- If the ground-water flow mainly occurs below the drains (viz. a high K_2D value), a variation in q, K_2 or h will result in a variation in the drain spacing L which is proportional to the square root of the variation in q, K_2 or h. For example, a variation of $+100\,\%$ in q, K_2 or h will result in a difference of about 40 % in the drain spacing ($\sqrt{2} = 1.41$); a variation of $+50\,\%$ will produce a difference of about 25 % and a variation of $-50\,\%$ will give a difference of 30 % ($\sqrt{0.50} = 0.70$). It can be seen from the formula that the drainage coefficient q has relatively the same effect on the calculation of the drain spacing as the hydraulic conductivity K.

  But the q- and K-values differ extensively in their importance as a source of possible errors in the calculation of the drain spacing required. The q-values, relating to the rainfall discharge or the leaching required to prevent salinization in irrigation projects, do not usually show much variation, and normally a rough calculation of the q value does not differ very much from a carefully computed value. On the other hand, the possible variations in the K-factor are more than 100-fold and the estimate of this factor may be extremely incorrect. There may also be a considerable variation in the depth of the impermeable layer.

  The q-values can be calculated approximately and discussed, but the hydrological soil factors K, D cannot be calculated, but have to be determined in the field *after* the various hydrological soil units have been outlined.

  The above-mentioned difference in the calculation of q- and KD-values might explain the fact that the importance of the q-value is often over-emphasized and that of the KD-values often underrated, especially for the layers below the drains.

- Finally, if we compare the formula

$$L^2 = \frac{4K\,[(D+h)^2 - D^2]}{q},$$

which was developed independently in the U.S.A. by DONNAN (1946), with HOOGHOUDT's formula, it will be found that DONNAN's formula is the same as the simplest form of HOOGHOUDT's formula, viz. when for a homogeneous soil ($K_2 = K_1$) only a horizontal flow is assumed ($d = D$), or the impermeable layer is in the vicinity of the drains):

$$4\,K\,[(D+h)^2 - D^2] = 4\,K\,[(D+h+D)\,(D+h-D)] = 4\,K\,(2D+h)\,h = 8\,K\,(D+0.5h)\,h = 8\,KDh + 4\,Kh^2$$

The essential feature of HOOGHOUDT's extensive analysis and calculations is that the ground-water flow occurring can be schematized as a horizontal flow up to a distance of 0.7 D from the drains (making the width of this zone L – 1.4 D) and a radial flow from a distance of 0.7 D up to the drain. He then specifies that the sum of the horizontal resistance (R_h) and the radial resistance (R_r) should be equal to the horizontal resistance in an equivalent layer having a thickness d, calculated over the entire length L (Fig. 4).

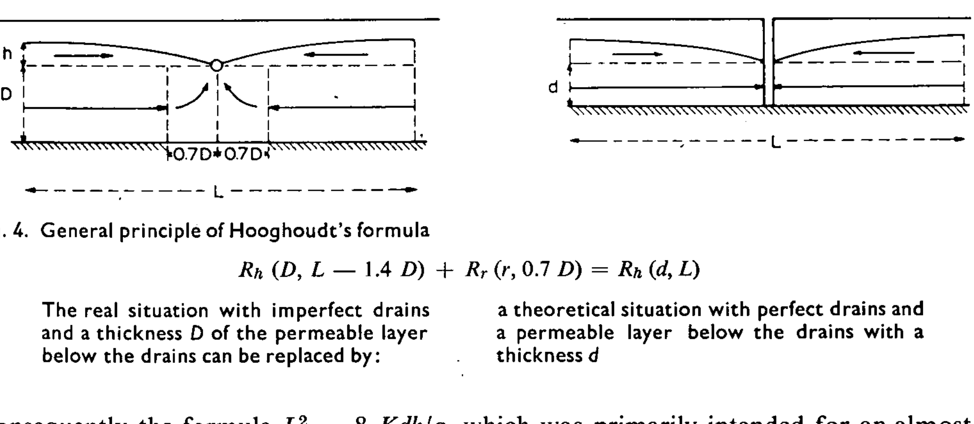


Fig. 4. General principle of Hooghoudt's formula

$$R_h\,(D, L - 1.4\,D) + R_r\,(r, 0.7\,D) = R_h\,(d, L)$$

The real situation with imperfect drains and a thickness D of the permeable layer below the drains can be replaced by: a theoretical situation with perfect drains and a permeable layer below the drains with a thickness d

Consequently the formula $L^2 = 8\,Kdh/q$, which was primarily intended for an almost entirely horizontal flow, can be applied to cases in which the radial resistance cannot be neglected. When a simplified formula is used for the radial resistance, this d value is easy to calculate, which makes it simpler to grasp any further explanation required (see Appendix A).

3.2. THE USE OF THE HOOGHOUDT FORMULA

The HOOGHOUDT formula is a very simple one, but calculations with this formula entail a laborious trial and error process. This is illustrated by the following example.
Given: a soil with an impermeable layer 3 m below the drain level ($D = 3$ m); $K_2 =$ 1 m/day; $K_1 = 0.5$ m/day; $q = 0.005$ m/day; $h = 0.60$ m; $r = 0.10$ m. Determine the proper drain spacing.

Let us assume that $L = 40$ m. From appendix B (d-values, HOOGHOUDT) for $D = 3$ m, $L = 40$ m and $r = 0.10$ m, is found $d = 2.16$ m:

Hence

$$L^2 = \frac{8 \times 1 \times 2.16 \times 0.6}{0.005} + \frac{4 \times 0.5 \times 0.36}{0.005}$$

$$1600 = 2080 + 144 = 2224 \text{ or } L > 40 \text{ m}$$

Then try $L = 50$ m with $d = 2.29$

$$2500 = \frac{8 \times 1 \times 2.29 \times 0.06}{0.05} + 144 = 2200 + 144 = 2344 \text{ or } L < 50 \text{ m}$$

Next try $L = 45$ m with $d = 2.23$:

$$2025 = \frac{8 \times 1 \times 2.23 \times 0.6}{0.05} + 144 = 2140 + 144 = 2280 \text{ or } L > 45 \text{ m}$$

Finally $L = 48$ m with $d = 2.26$ m:

$$2304 = \frac{8 \times 1 \times 2.26 \times 0.6}{0.05} + 144 = 2178 + 1.44 = 2322$$

This is close enough to the 48 m assumed. Hence the theoretically required spacing is about 48 m.

If it is desired to know whether the drain spacing required would differ greatly with a different depth to the impermeable layer, e.g. 2 or 4 metres, we should have to start over again with a new set of calculations. In order to simplify the work of calculation a nomograph has been prepared.

3.3. Nomograph I for tile drains ($r = 0.10$)

This type of nomograph was first compiled by W. H. van der Molen with the aid of nomographic calculations for one specific value of q (7 mm) and for a number of values of h. The nomograph presented here is more widely applicable as it can be used for all values of q and h. It was constructed by a graphical method.

Although tile drains may have a different radius, only one tile drain nomograph is given here, viz. for $r = 0.10$ m, the reason being that the variations that occur in tile drains are relatively slight and therefore have little effect on the calculated drain spacing. Moreover, except for sodic soils, the backfilled trench also has a favourable effect on the radial resistance that occurs. But this effect is difficult to assess, so that it is assumed that a nomograph for $r = 0.10$ can be regarded as an acceptable value under the conditions occurring in practice. For practical reasons the nomograph has been split up in two parts: Graph Ia for values L = 5– 25 m

Ib „ „ L = 10–100 m

To illustrate the use of graph I, the same example will be used:

Given:	*Calculation*:	
$q = 5$ mm/day	$\frac{8h}{q} = 960$	$\frac{4h^2}{q} = 290$
$h = 0.60$ m		
$K_2 = 1$ m/day	$K_2 = 1$	$K_1 = 0.5$
$K_1 = 0.5$ m/day		
$D = 3$ m	$\frac{8K_2h}{q} = 960$	$\frac{4K_1h^2}{q} = 145$
$r = 0.10$ m		

Connect point 960 on the left-hand side of graph Ib to point 145 on the right-hand side. The point of intersection of this line and the line for $D = 3$ gives the required drain spacing L. In the present instance this is approximately 47.5 m.

It will be seen that the effect of the depth of the impermeable layer can be read immediately from the graph. In the absence of an impermeable layer the spacing would have been 69 m. If this layer had been at a depth of 5 m ($D = 5$) the spacing would have been approximately 57 m, whereas $D = 1$ m gives a value of L of about 32 m.
The effect of the K-factor can also be very rapidly ascertained. Had the hydraulic conductivity above the drains K_1 been 1 m instead of 0.5 m/day (960–290), the drain spacing when $D = 3$ m would have been about 49 m, instead of 48 m as in the former case. But if K_2 were 0.5 instead of 1 m/day (480–145), the drain spacing when $D = 3$ m would be 34 m instead of 48 m.

This example shows that the depth of the impermeable layer D and the K_2-value have a very great effect on the drain spacing. This example also demonstrates the fact that in this case the hydraulic conductivity of the layer above the drains K_1 has very little effect, so that provided no dense layers occur there is no need to make a careful investigation of this factor and an estimate is sufficient. But in other cases (viz. when D and K_2 are small and h and K_1 are large) the factor K_1 may be very important.
Usually a great number of K determinations have to be made in field surveys for tile drainage. The K-values found may vary extensively, especially when they are determined by means of a grid system and not on the basis of hydrological soil units previously distinguished. The question now arises: to what extent and in what manner should these varying K-values be averaged for the purpose of determining the required drain spacing? In order to answer this question one should realise that, with a fixed value of q and h for a particular tile drainage project, the drain spacing is determined by the combination of K_1, K_2 and D. The K_2-values should be averaged only when there is practically no difference between the corresponding values of K_1 and D. In other cases it is advisable to determine the corresponding drain spacing at each point of the investigation. This can be done very rapidly because $8h/q$ and $4h^2/q$ are fixed values and it is possible to make use of a slide-rule and nomograph. The resultant L-values can then be averaged in groups. In most cases the drain spacings arrived at will differ extensively within the same project. For practical reasons, however (design and maintenance), it is advisable to adhere to a single drain spacing within given blocks (e.g. fields). No hard-and-fast rule can be given for the proper delimitation of these drainage units.

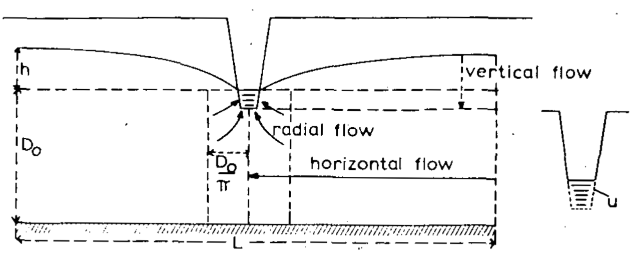
4. NOMOGRAPHS BASED ON ERNST'S FORMULA

THE SOIL BELOW THE DRAINS CONSISTS OF ONE OR TWO LAYERS
IMPERMEABLE LAYER AT A RELATIVELY SHALLOW DEPTH ($D < {}^1/_4\, L$)

4.1. EXPLANATION OF ERNST'S FORMULA

The principle underlying ERNST's formula may be described as follows. The flow is divided into the following three components, a vertical (v), a horizontal (h) and a radial flow (r) (Fig. 5). A given drain spacing being assumed, the hydraulic head required for this flow is calculated for each component with the aid of a given formula. When the sum of the three hydraulic heads required is equal to the total available head ($h = h_v + h_h + h_r$) the result is the drain spacing required. Hence, this formula involves the same kind of trial and error process as does the Hooghoudt formula.

Fig. 5. General principle of the formula of Ernst



ERNST's general formula is as follows:

$$h = q\frac{D_v}{K_1} + \frac{qL^2}{8KD} + \frac{qL}{\pi K}\, ln\frac{D_o}{u}$$
$$= h_v + h_h + h_r$$

The following most frequently occurring situations can be distinguished:

- Homogeneous soils with an impermeable layer at a depth D_o below the level of the drains: $D_o < {}^1/_4 L$ (Fig. 6).
- The level of the drains coincides with the interface of two layers of varying permeability (Fig. 7).
- The soil below the drains consists of two different layers (Fig. 8).

In connection with the formula the following remarks may be made:

- h = *available hydraulic head* = difference between the level of the water in the drains and midway between the drains. The notation Δh, viz. the difference between two

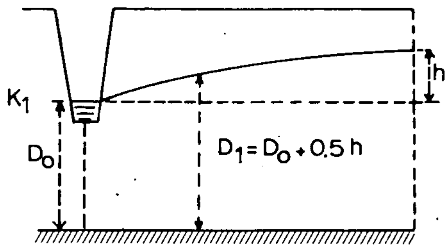


Fig. 6. Homogeneous soil $D_o < {}^1/_4 L$

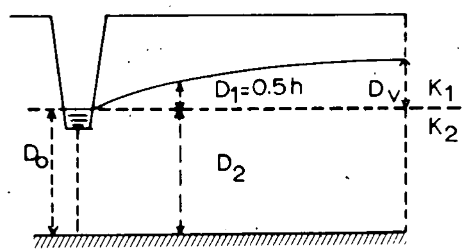
$$h = \frac{qL^2}{8K_1D_1} + \frac{qL}{\pi K_1} \ln \frac{D_o}{u}$$

Fig. 7. Two soil layers. Water level in the drain coincides with boundary K_1/K_2

$$K_1 \gtrless K_2 : h = \frac{qL^2}{8(K_1D_1 + K_2D_2)} + \frac{qL}{\pi K_2} \ln \frac{D_o}{u}$$

$$K_1 << K_2 : h = q\frac{D_v}{K_1} + \frac{qL^2}{8K_2D_2} + \frac{qL}{\pi K_2} \ln \frac{D_o}{u}$$

$K_1 >> K_2$: use formula Hooghoudt



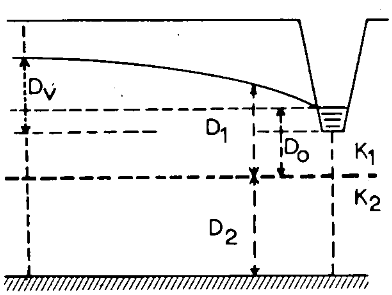


Fig. 8. Two soil layers. Drains entirely in the upper layer

$$h = q\frac{D_v}{K_1} + \frac{q\,L^2}{8\,(K_1D_1 + K_2D_2)} + \frac{qL}{\pi K_1} \ln \frac{a\,D_o}{u}$$

levels, would be more correct in this case and is also employed by ERNST (1954, 1962). For practical reasons, however, the notation h is used here.

- D_v = thickness of the layer for the *vertical component*. The upper limit of this layer invariably coincides with the level of the water midway between the drains, whereas the lower limit varies somewhat according to the position of the drains with respect to layers of different permeability.
When the drains are located entirely in the uppermost layer, the vertical flow is taken into account up to the bottom of the drains (Fig. 8). If the drain is on the boundary between layers of poor and good permeability, D_v is calculated up to this boundary (Fig. 7).
In most cases the vertical component is small and may be ignored. For instance, when $q = 0.005$ m, $D_v = 0.6$ m and $K_1 = 0.3$ m, then $h_v = q\ (D_v/K_1) = 0.005 \times 0.6/0.3 = 0.01$ m. But if we have a layer of very poor permeability ($K_1 = 0.02$ m) located on a layer of very good permeability (e.g. heavy basin clay on a sandy river deposit) and for instance $q = 0.010$ and $D_v = 0.60$, then $h_v = 0.010 \times 0.60/0.02 = 0.30$ m, so that h_v should not be neglected.
- KD, viz. without index, denotes the product, or sum of the products of the permeability (K) and thickness (D) of the various layers for the *horizontal component*; this has to be specified in further detail according to the hydrological situation (K_1D_1 or K_2D_2 or $K_1D_1 + K_2D_2$).
- D_2 = thickness of the second layer for the *horizontal component*; this is a given magnitude determined in the field.
When the level of the drains coincides with the interface of two layers, then $D_2 = D_o$ (Fig. 7).
- D_1 = the average cross section for the *horizontal component* in a layer of permeability K_1. The thickness of this layer cannot be directly inferred from the field data but, where important, will have to be calculated. Depending on the position of the drains with respect to the interface of two layers of different permeability (K_1 and K_2), $D_1 = 0.5\ h$ (Fig. 7) or $D_1 = \dfrac{D_o + (D_o + h)}{2} = D_o + 0.5\ h$ (Fig. 6 and 8), wherein D_o is the thickness of the layer for the radial component.
- $ln\ (D_o/u)\ /\ \pi K = R_r$ = *radial resistance*. In this case $K = K_1$ or K_2, depending on the position of the drains.
- D_o = thickness of the layer for which the *radial resistance* is calculated. The upper limit of this layer invariably coincides with the level of the water in the drains, whereas the

lower limit is formed by an impermeable layer or a layer of differing permeability.
- u = *wetted perimeter* = bottom of the ditch + twice the height of the water in the ditch. Theoretically it would be necessary to take into account the gradient of the slope of the ditch, but such a degree of accuracy has little practical value. For comparison with other formulas, $u = \pi r$.

4.2. Ernst's and Hooghoudt's formulas compared

If we now compare Ernst's formula with Hooghoudt's, the following differences and correspondences may be observed:

- The component for the vertical flow is disregarded by Hooghoudt. In most cases, however, this component is small and may be neglected.
- The horizontal flow or resistance is calculated by Hooghoudt over the length L–1.4 D (Fig. 4), whereas Ernst calculates it over the entire drain distance L (Fig. 5).
  As a result the horizontal resistance calculated by Ernst is greater than that calculated by Hooghoudt.
- The value calculated by Ernst for the radial resistance is, however, smaller than that calculated by Hooghoudt (Ernst: $\frac{1}{\pi K} \ln \frac{D_0}{u} = \frac{1}{\pi K} \ln \frac{D_0}{\pi r} = \frac{1}{\pi K} \ln \frac{0.32 D_0}{r}$ and Hooghoudt: $\frac{1}{\pi K} \ln \frac{0.70 D_0}{r}$).

In most cases the sum of the horizontal and radial resistances calculated according to Ernst's formula is substantially the same as the result of Hooghoudt's formula. Consequently both formulas usually give practically the same drain spacings. It is only when K_1 is much greater than K_2 ($K_1 >> K_2$) that, unlike Hooghoudt's formula, Ernst's formula gives a slightly smaller drain spacing. According to Ernst (1963, p. 35) no acceptable formula has been found for the radial resistance for this special case. Ernst's formula may be employed for this case but without further investigation it is impossible to say what degree of accuracy will be obtained. This depends, for instance, on the position of the drains with respect to the interface of the various layers.

The advantages of the nomograph based on Hooghoudt's formula (Graph I) are the fairly high speed of calculation and the possibility of rapidly ascertaining the relative effect of the different factors influencing the drain spacing. But the drawback of HOOGHOUDT's method is that in order to retain these advantages a separate nomograph has to be compiled for each value of the drain radius (r). Consequently ERNST's drainage formula is employed for the nomographs for ditches with various r values. A further advantage of this formula is that it enables drain spacing to be calculated when the soil *below* the drains is non-homogeneous, consisting of two different layers.

4.3. NOMOGRAPHS II AND IIA FOR TILE DRAINS AND DITCHES

These nomographs will now be explained with reference to examples of calculating the following most frequently occurring situations:

4.3.1. HOMOGENEOUS SOIL

$D_o < {}^1/_4 L$ (Fig. 6) Use graph II

$$h = \frac{qL^2}{8K_1D_1} + \frac{qL}{\pi K_1} \ln \frac{D_o}{u}$$

Given:	*Calculations*:
$q = 0.005$ m/day	$D_1 = D_o + 0.5\,h = 3 + 0.30 = 3.30$
$h = 0.60$ m	$KD = K_1D_1 = 0.5 \times 3.30 = \mathit{1.65}$
$K_1 = 0.5$ m/day	$h/q = 0.600/0.005 = \mathit{120}$
$D_o = 3$ m	$D_o/u = 3/0.75 = 4$
$u = 0.75$ m	Read on bottom of the graph II: $\frac{1}{\pi} \ln \frac{D_o}{u} = 0.44$
	$R_r = 0.44/K_1 = 0.44/0.5 = 0.88$

Connect points 1.65 on the left to point 120 on the right. Where this line intersects the line for $R_r = 0.88$, the L reading is 34 m.
When $K_1 = K_2$, the use of the graph can be somewhat simplified by using the values for D_1, $\ln (D/u) / \pi$ and hK/q, or, in the present case respectively: 3.30 – 0.44 – 60 $\rightarrow L = 34$.

4.3.2. THE LEVEL OF THE DRAINS COINCIDES WITH OR IS BELOW THE INTERFACE OF TWO LAYERS OF VARYING PERMEABILITY (Fig. 7)

$$h = \frac{qL^2}{8\,(K_1D_1 + K_2D_2)} + \frac{qL}{\pi K_2} \ln \frac{D_o}{u}$$

Since HOOGHOUDT's formula can be used for this situation, as well as for the case discussed under 4.3.1., the same data will be employed as in Section 3.3, pag. 21. This will also permit comparison of the two formulas.

Given:	*Calculations*:
$q = 0.005$ m/day	$D_1 = 0.5\,h = 0.30$ m
$h = 0.60$ m	$KD = K_1D_1 + K_2D_2 = 0.5 \times 0.3 + 1 \times 3 = 3.15$
$K_1 = 0.5$ m/day	$h/q = 0.600/0.005 = 120$
$K_2 = 1$ m/day	$D_o/u = 3/0.30 = 10 \rightarrow 0.73$
$D_o = D_2 = 3$ m	$R_r = 0.73/K_2 = 0.73/1 = 0.73$
$u = 0.30$ m	Read on graph II: 3.30 – 0.73 – 120 $\rightarrow L = 47$ m.

The following checking calculations may be used in order to illustrate ERNST's formula:

$$h = h_h + h_r$$

$$= \frac{qL^2}{8\,(K_1D_1 + K_2D_2)} + qL\frac{1}{\pi K_2}\, ln\frac{D_o}{u}$$

$$= \frac{0.005 \times 47^2}{8 \times 3.15} + 0.005 \times 47 \times 0.73$$

$$= 0.44 + 0.17 = 0.61,$$

which is near enough when compared with the given value of h (0.60). Had no graph been available a trial and error procedure would have been necessary, involving many calculations (L = 50, L = 40, etc.), or else L would have to have been calculated from the equation:

$$\frac{0.005}{8 \times 3.15} L^2 + 0.005 \times 0.73\, L - 0.60 = 0$$

$$\left(\text{for } aL^2 + bL - c = 0,\ L = \frac{-b + \sqrt{b^2 + 4\,ac}}{2a}\right)$$

Use of graph II for h/q values > 300 and $R_r > 3$

There are cases in which graph II cannot be used without further calculations. The highest value for h/q shown in the graph is 300 and the highest value for R_r is 3. When, for instance, $h/q = 800$, $KD = 20$ and $R_r = 1.8$, connect point $800/4 = 200$ to point $20/4 = 5$. Where this line intersects the line for $R_r = 1.8$ the L' reading is 60.
In this case $L = 4L' = 4 \times 60 = 240$ m.
Point $800/8 = 100$ and $20/8 = 2.5$ would, of course, give the same result, viz. $L = 8 \times 30 = 240$ m.
For the exceptional cases of $R_r > 3$, divide the R_r value and h/q values by 2 and multiply the KD value by 2. For example, the reading of 3 – 4.2 – 200 is the same as 6 – 2.1 – 100.

4.3.3. Soil below the drain consists of two different layers (Graphs II and IIa, Fig. 8)

This situation is rather more complex than the previous cases discussed, and therefore requires more calculation and an auxiliary nomograph (IIa) for determining the radial resistance. In order to construct this auxiliary nomograph Ernst made use of the relaxation method.
The calculations required and the use of graph II in combination with graph IIa will be illustrated by the following example.

Given:

$q = 0.010$ m/day
$h = 1.20$ m
$D_o = 0.60$ m
$u = 0.50 + 2 \times 0.20 = 0.90$ m
$D_2 = 3$ m
$K_1 = 0.2$ m/day
$K_2 = 2$ m/day

Calculations:

$D_v = 1.20 + 0.20 = 1.40$
$h_v = q\,(D_v/K_1) = 0.010 \times 1.40/0.20 = 0.07$ m
$h_h + h_r = h - h_v = 1.20 - 0.07 = 1.13$
$h/q = 1.13/0.010 = 113$
$D_1 = D_o + 0.5\,h = 0.60 + 0.60 = 1.20$
$KD = K_1D_1 + K_2D_2 = 0.2 \times 1.20 + 2 \times 3 = 6.24$
Radial resistance graph IIa:
$K_2/K_1 = 2/0.2 = 10$
$D_2/D_o = 3/0.60 = 5.$
Read on graph IIa: $a = 4.3$
$aD_o/u = 4.3 \times 0.60/0.90 = 2.9 \rightarrow$ (Graph II)

$$\frac{1}{\pi}\, ln\, \frac{D_o}{u} = 0.34$$

$R_r = 0.34/K_1 = 0.34/0.20 = 1.7$

Using graph II: $KD = 6.2$; $R_r = 1.7$; $h/q = 113 \rightarrow L = 43$ m.

In the above situation, in which $K_2D_2 >> K_1D_1$, the calculations could be simplified by taking into account only the flow below the drains:

$KD = K_2D_2 = 2 \times 3 = 6$ m
$h/q = 1.20/0.010 = 120$
$R_r = 1.7$
6 — 1.7 — 120 $\rightarrow$ 45 m, which is satisfactory.

If, instead of the above case, the drains would reach into the deeper, permeable layer (D_2), the drain distance can be increased significantly. Using the simplified calculations ($KD = K_2D_2$) and graph II the following result is obtained:

$h \quad = 1.20 + 0.60 = 1.80$
$h/q \quad = 1.80/0.010 = 180$
$KD \quad = K_2D_2 = 2 \times 3 = 6$
$D_o/u \quad = 3/0.90 = 3.3 \rightarrow 0.38$
$R_r \quad = 0.38/2 = 0.19$
6 — 0.19 — 180 $\rightarrow L = 88$ m

Hence, in this case, the depth of the drains is an extremely important factor.

5. NOMOGRAPH FOR HOMOGENEOUS SOIL WITH IMPERMEABLE LAYER AT GREAT DEPTH ($D > 1/4\ L$) (Graph III)

Several solutions are known for this special case (HOOGHOUDT 1940, GUSTAFSON 1946, VAN DEEMTER 1950, ERNST 1954, 1962, VAN SCHILFGAARDE, KIRKHAM and FREVERT 1961, TOKSÖZ and KIRKHAM). If we leave out of account situations in which the water table is of a relatively marked convex shape ($h > L/5$) the following simple formula is found to be applicable:

$$h = \frac{qL}{\pi K} \ln \frac{L}{u}$$ (ERNST 1954, TOKSÖZ and KIRKHAM 1961).

This formula includes only a radial resistance, the reason being that, in this case, the horizontal and vertical resistances are negligible compared with the radial resistance. Nomograph III provides a rapid answer about the maximum drain spacing possible for given values of q, h, K and u, it being assumed that $D > 1/4\ L$.
This nomograph will be illustrated by the following example:

$q \;\; = 0.002$ m/day
$h \;\; = 1.2$ m
$K \;\; = 0.8$ m
$u \;\; = 1.50$ m
$h/q = 1.2/0.002 = 600$

Connect point 600 to $K = 0.8$. Where this line intersects the vertical line $L \ln \frac{L}{u}$, move in a horizontal direction and at the point where the line for $u = 1.50$ is met, read $L = 270$.

The influence of the wetted perimeter u of the ditch can also be rapidly and easily ascertained. In this case when $u = 0.30$, L would be 215 m, and $u = 3$ m would give $L = 310$ m, etc.

If it is not known whether in a given instance we have a situation in which $D < {}^1/_4\,L$ or $D > {}^1/_4\,L$, graph II or graph III may be used. If it is found that $D > {}^1/_4\,L$, graphs II and III will give the same result.

Example:

$q = 0.010$ m/day	Graph II: $D = 9$ m; $D_o/u = 9/0.30 = 30 \rightarrow 1.08$; $hK/q = 40$;
$h = 0.80$ m	$9 - 1.08 - 40 \rightarrow L = 28$ m
$K = 0.5$ m/day	
$D = 9$ m	Graph III: $h/q = 80$, $K = 0.5$, $u = 0.30$; $L = 27$ m
$u = 0.30$ m	

6. A NOTEBOOK METHOD FOR COMPUTING DRAIN SPACINGS[1])

MODIFIED FORMULA OF ERNST

(Graph IV)

In HOOGHOUDT's drainage formula $L^2 = 8Kdh/q$, the radial resistance is taken into account by a correction of the layer D by means of the 'equivalent' layer d, in which $d < D$. Generally speaking, however, this correction can also be applied to the drain spacing L. Some years ago MOODY supplied the formula $D \ ln \ D/4_r$ for this type of correction (MAASLAND 1956, DUMM 1960). In most cases this correction proved adequate, but in certain others it was unsatisfactory. This was to be expected, since the correction is partly dependent on the drain spacing L. This difficulty has now been solved by means of the following calculations which are based on the ERNST formula:

Given:

$$\frac{L^2}{8KD} = \frac{L_0^2}{8KD} - \frac{L}{\pi K_2} \ln \frac{D_0}{u}$$

in which

L = the correct drain spacing, based on both the horizontal and radial resistance,
L_0 = the drain spacing when only the horizontal resistance is included.

Problem: Calculate L when all other necessary data are known.

[1]) This method is termed 'Notebook Method' in order to give it a designation and because the graph it requires can be placed in a notebook.

Solution:

$$\frac{L^2}{8KD} = \frac{L_o^2}{8KD} - \frac{L}{\pi K_2} \frac{D_o}{u}$$

multiplying by $\dfrac{8KD}{LL_o}$ gives $\dfrac{L}{L_o} = \dfrac{L_o}{L} - \dfrac{8KD}{\pi K_2 L_o} \ln \dfrac{D_o}{u}$;

assuming $\dfrac{KD}{K_2 L_o} \ln \dfrac{D_o}{u} = a$, we obtain $L/L_o = L_o/L - 2.55\ a$.

For the different a-values, graph IV shows the corresponding values of L/L_o. Since L_o is known, L can now be calculated: $L = L/L_o \times L_o$

It can be seen from the graph and calculations that when $a < 0.4$ or $(KD/K_2) \ln (D_o/u) < 0.4\ L_o$ (which is usually the case) the equation is, with an error of not more than 2% to 3%, as follows:

$$L = L_o - (KD/K_2) \ln (D_o/u),$$

or for a homogeneous soil: $L = L_o - D \ln (D_o/u)$

or: $L = L_o - 2.3\ D \log (D_o/u)$

If moreover only the flow below the level of the drains is taken into account, then $D = D_o$.

In order to illustrate the use of the modified formula and graph IV, the same example will be used as that given in the preceding chapters.

Given:

$q = 0.005$ m/day
$h = 0.6$ m
$K_2 = 1$ m/day
$K_1 = 0.5$ m/day
$D = 3$ m
$u = 0.30$ m

Calculation:

$$KD = 1 \times 3 + 0.5 \times 0.3 = 3.15$$

$$L_o^2 = 8KDh/q = 8 \times 3.15 \times 0.6 \times 1000/5 = 3024$$

$$L_o = 55 \text{ m}$$

$$D_o/u = 3/0.3 = 10;\ ln\,(D_o/u) = 2.3;\ \ (KD/K_2)\,ln\,(D_o/u) = \frac{3.15}{1} \times 2.3 = 7.2$$

$$a = \frac{7.2}{55} = 0.13; \text{Graph IV: for } a = 0.13, \text{ read: } \frac{L}{L_o} = 0.85$$

$$L = 0.85 \times 55 = 47 \text{ m}$$

Since, in this case, $D\ ln\ \frac{D_o}{u} < 0.4\ L_o$ and the K_1D_1 value has little effect, the following simpler calculation would be sufficiently accurate:

$$L_o^2 = 8 \times 3 \times 0.6 \times 1000/5 = 2880$$

$$L_o = 54$$

$$L = L_o - D\ ln\,(D_o/u) = 54 - 3 \times 2.3 = 47 \text{ m.}$$

This method of calculation can also be used when the impermeable layer is at an infinite depth, since for D-values greater than $^1/_4\ L$, the decrease in the calculated horizontal resistance, is about equal to the increase of the calculated radial resistance.

Example:

$q\ =\ 0.002$ m/day
$h\ =\ 1.20$ m
$K = 0.8$ m/day
$u\ =\ 1.50$ m

Estimate L and take a value of D which is greater than $^1/_4\ L$ but smaller than L; this can be checked after the calculation has been concluded. In this case, for example, $D = 120$ m.

$$L_o^2 = \frac{8 \times 0.80 \times 120 \times 1.2 \times 1000}{2} = 461.000 = 46 \times 10^4$$

$$L_o = 680 \text{ m}$$

$D/u = 120/1.50 = 80;\ ln\ 80 = 4.37;\ D\ ln\,(D/u) = 120 \times 4.37 = 525\ \ a = 525/680 = 0.78 \rightarrow L/L_o = 0.42$ and $L = 0.42 \times 680 = 286$ m

This shows reasonably good agreement with the calculation of L by an other method in Section 5, page 31 ($L = 270$ m).

7. NOMOGRAPH FOR NON-STEADY FLOW

IRRIGATION PROJECTS. GLOVER/DUMM FORMULA

(Graph V)

During the irrigation of a field or the leaching of saline soils there is considerable infiltration of water in a short period. As a result, there is a rapid rise in the ground water which gradually falls again over a period of days. In this case, however, the premises are not a given intensity of precipitation and a given water table (combination of q and h), but the fall in the water table required over a given number of days, starting from a given initial situation (fall from h_o to h_t over a period of time t) (see Fig. 9).

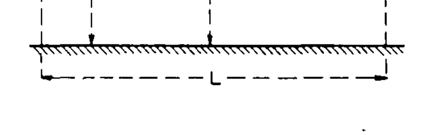


Fig. 9. Falling water table or transient flow; $D < {}^1/_4 L$

$$\frac{h_t}{h_o} < 0.8, \text{ then } h_t = h_o\, 1.16\, e^{-t/j}$$

$$j = \frac{VL^2}{10KD}; \qquad D = D_o + \frac{h_o + h_t}{4}$$

$$L^2 = \frac{10\,KDt}{V\ln\left(1.16\,\frac{h_o}{h_t}\right)}$$

Consequently the formulas employed for this purpose do not include the drainage coefficient q. If a given amount of water has to be drained off, say 20 mm in 10 days, this is converted into an equivalent fall required at the midpoint height between the drains.

A further difference is that in the non-steady or transient flow formula both the KD-value and the drainable pore space V have to be known. Other terms for this are volume fraction of pores drained, specific yield, effective porosity, etc. The magnitude of this factor is not constant for a given soil layer but depends, for example, on the distance between this layer and the ground-water level. Hence the value employed is in the nature of an average value. If no measurements of this factor are available, then in the opinion of the present author they can be estimated in most cases by employing the formula $V = \sqrt{K}$ in which V is expressed in ratios by volume and K in cm/day, e.g. $K = 100$ cm, $V = \sqrt{100} = 10\%$; $K = 10$ cm, $V = 3\%$. In the non-steady flow formula given here $V = 10\%$ is written, in the customary manner, as $V = 0.10$.
The simple nomograph given here is based on a formula employed by GLOVER/DUMM in a recent article (DUMM 1960), together with two new concepts termed 'reservoir coefficient' and 'tail recession' by KRAIJENHOFF VAN DE LEUR (1958). These two concepts are very useful for the understanding of the non-steady nature of the ground-water flow in question.

In an earlier article GLOVER/DUMM (1954) assumed the water table between the drains to be a horizontal surface at the start of each drainage cycle. But since a flat water table is not likely to occur at the beginning of the drainage period following the installation of parallel drains, a similar formula has been used which was obtained by assuming that the initial phreatic surface has the shape of a parabola of the 4th degree (DUMM 1960). Since the old formula has no special advantages as regards the calculations or the construction of a nomograph, and the use of the parabolic shape constitutes a worthwhile advance in the application of the theory, the nomograph given here is based on the recent formula viz.:

$$\frac{h_t}{h_o} = \frac{192}{\pi_3} \sum_{n=1,3,5}^{\infty} (-1)^{\frac{n-1}{2}} \frac{n^2 - 8/\pi^2}{n^5} \exp\left(-\frac{\pi^2 n^2 KDt}{VL^2}\right)$$

An approximate solution is obtained by discarding all but the first term (using only n = 1), and introducing

$$\frac{VL^2}{\pi^2 KD} = j$$

we obtain

$$h_t/h_o = 6.2\,(1 - 0.813)\,e^{-t/j}$$
$$= 1.16\,e^{-t/j}$$

where j is the *reservoir coefficient* and $e = 2.72$ the base of the natural logarithm.
The reservoir coefficient is expressed in days (KRAIJENHOFF VAN DE LEUR 1958)[1] and characterises the drainage situation by incorporating the chief hydrological properties (KD-value, L and V, but not the radial resistance). If the j-value is high (KD small, and/or V and L large) the water table will fall slowly, whereas a low j-value causes a rapid draw-down.

To give a concrete example:

$$V = 0.05,\ L = 100 \text{ m},\ K = 0.25 \text{ m and } D = 2 \text{ m}.$$

$$j = \frac{VL^2}{\pi^2 KD} = \frac{0.05 \times 100^2}{10 \times 0.25 \times 2} = 100 \text{ days}$$

It can be seen from graph V that the water table will drop halfway ($h_t/h_o = 0.5$) when $t/j = 0.84$, or after $0.84j$ days.
Hence if $j = 100$ days, this will take 0.84×100, or about 84 days, whereas in another drainage situation in which, for instance $V = 0.05$, $L = 30$ m, $K = 1.5$ m, $D = 3$ m: $j = (0.05 \times 30^2) / (10 \times 1.5 \times 3) = 1$, this will take 0.8 days.
Strictly speaking, the above formula $h_t/h_o = 1.16\,e^{-t/j}$ is only valid when the sum of the following terms in the series becomes negligibly small. As this is an alternating series with decreasing terms, so that the absolute value of the second term is greater than the sum of all subsequent terms, it may be assumed that the sum of these subsequent terms may be neglected when the absolute value of the second term is, say, less than 1% of the first term. The absolute value of this second term is relatively highest when $e^{-t/j}$ is relatively small. This is the case when t/j is small, i.e. when the water table is beginning to draw down.
When the second term ($n = 3$) is equal to 1 per cent of the first term ($n = 1$) the equations are:

$$\frac{9 - 8/\pi^2}{243}\,e^{-9t/j} = 0.01\,\left(1 - \frac{8/\pi^2}{1}\,e^{-t/j}\right)$$

1) $$\frac{L^2}{KD} \rightarrow \frac{m^2}{m/days \times m} = \text{days}$$

$$0.0337\, e^{-9t/j} = 0.01 \times 0.187\, e^{-t/j}$$
$$e^{8t/j} = 3.37/0.187 = 18 \text{ or } 8^{t/j} = 2.89 \text{ or } t/j = 0.36.$$

In the same way it can be calculated that, neglecting the second term in the series, gives a 2% difference for $t/j = 0.27$, 5% for $t/j = 0.16$ and 10% for $t/j = 0.07$. In the nomograph these differences are corrected by the curve diverging from the straight line from $h_t/h_o = 0.8$.

The following observations may be made with reference to the nomograph and the non-steady-state formula:

- When the water table begins to fall there is no definite ratio between the level of the ground water h and the ground-water outflow qL. Compared with the subsequent fall the initial fall is relatively slow. It can be seen from the nomograph that 0.38 j days are required for the first 20% drop. Subsequently only 0.22 j days are required for a 20% drop (h_t/h_o: 0.8 → 0.64, 0.5 → 0.4, etc.). After $h_t/h_o = 0.85$ to 0.80 or $t/j \approx 0.3$ to 0.4 the 'tail recession' occurs (KRAIJENHOFF VAN DE LEUR, 1958) in which there is a fixed ratio between R (the storage or phreatic ground water), qL (the rate of flow from two sides into a unit length of drain) and h. Moreover, $R = j \times qL = (2/\pi)\, VhL$ and $q = 2\pi KDh/L^2$.
- These three tail recessions can be shown as straight lines on semi-log. paper, each having the same slope. The reservoir j coefficient determines the fixed ratio between the ground-water storage and the ground-water outflow.
- If the initial drop, e.g. from h_o to $h_t = 0.9\, h_o$, were to be calculated by means of the formula $h_t/h_o = 1.16\, e^{-t/j}$, the time calculated for this would be greater than that actually needed.
  There would be a still greater difference had the calculation been made with DUMM's original formula, based on a flat initial water table, which has the coefficient $4/\pi = 1.27$ instead of 1.16.
  Moreover if the first drop were to be calculated on the basis of the steady state in which $h_t/h_o = 1.03\, e^{-t/j}$, the calculated drop would again be too rapid compared with the actual drop. The exact value for the first 10% to 20% lies somewhere between these two extremes, the coefficient 1.16 probably being a good approximation.
  During the further lowering (after $h_t = 0.8\, h_o$) the lines for the coefficients 1.03, 1.16 and 1.27 run parallel and therefore show the same unit of time for the subsequent lowering (20% drop 0.22 j days).

Consequently in the nomograph based on the formula $h_t/h_o = 1.16\ e^{-t/j}$ the initial lag in the lowering of the ground water is accounted for as well as possible. This lag is included in the reading of the time required for a drop to, say, $h_t = 0.5\ h_o$.

- In calculating the KD-value to be used in steady-state formulas it was assumed that D = the average cross section for horizontal flow $= D_o + 0.5\ h$. In the case of a falling water table $D = D_o + 0.5\ h$ average. But how is h_{av}. to be calculated in this case?
  The true course of $h\ (t)$ is intermediate between $h_t = h_o{}^{-at}$ and $h_t = 1.16\ h_o e^{-at}$, from which we could therefore give $h_{av} = 1.08\ h_o/at\ (1 - e^{-at})$ as a reasonably good approximation. In this connection it should be noted that an exact value of D_1 — the flow depth in the upper layer — is actually only important when most of the flow passes through the layer above the drains (D_o small, $K_1 > K_2$) which is not usually the case. Hence in order to calculate h_{av} it is advisable to employ the readily determined arithmetical mean or $D = D_o + 0.5\ h_{av} = D_o + 0.5\ (h_o + h_t)\ /\ 2 = D_o + (h_o + h_t)\ /\ 4$.
- The formula given here takes into account only the horizontal resistance and the reduction in spacing due to the flow restriction caused by the convergence of flow lines as they approach the drains, the radial resistance being ignored. The spacing calculated should therefore be corrected. This correction may be calculated in the manner indicated in Section 6 (Notebook Method). It should be noted that the correction given here is derived for a steady-state drainage formula but is here used in a nonsteady-state formula. This is probably justified as there are no reasons for assuming that the flow distribution for nonsteady-state conditions will differ greatly from the steady-state distribution (MAASLAND 1956, KRAIJENHOFF VAN DE LEUR 1962).

The use of the nomograph, V will be illustrated by the following examples:

Data given

h_o = maximum height of the water table = 1.20 m

Drainage requirement: draw-down of 0.60 m in 15 days

K = 0.25 m/day

V = 0.05

D_o = 5m

Problem: required drain spacing L

Calculation

$h_t \quad = h_o - 0.60 = 1.20 - 0.60 = 0.60$ m
$h_t/h_o = 0.60/1.20 = 0.5$; Graph V: $t/j = 0.84$ or $j = 15/0.84 = 17.9$
$D \quad = D_o + D_1 = D_o + (h_o + h_t) / 4 = 5 + 1.80 / 4 = 5.45$
$KD = 0.25 \times 5.45 = 1.36$

$$j = \frac{VL^2}{10\ KD} \text{ or } L^2 = \frac{10\ KD}{V} j$$

$$L^2 = \frac{10 \times 1.36}{0.05} \times 17.9 = 4860 \text{ or } L = 70 \text{ m}$$

Correction for radial resistance: When $u = 0.30$ m the following calculation is required:

$$D\ ln\ (D/u) = 5\ ln\ (5/0.30) = 5\ ln\ 17 = 5 \times 2.8 = 14 \text{ m},$$

which is less than $0.4\ L_0$. Hence no further correction is required and the drain spacing is $70 - 14 = 56$ m, or, for more practical purposes, about 50 m.

In the example given here the problem was to calculate the required drain spacing. But in other cases the drainage situation is given and the problem is to calculate the number of days in which the water table will fall to a given level.
The example given here may also serve to illustrate a possible combination of steady and nonsteady-state formulas.
A situation is assumed in which the waterlevel in the drains coincides with the boundary of K_1 and K_2 and conditions of the type prevailing in the Netherlands ($q = 7$ mm, drain depth 1 m, etc.):

Data given for calculating a steady-state formula:
$K_1 = 0.5$ m/day
$K_2 = 1$ m/day

Allowable maximum height of the water table: 0.50 m below ground level

Depth of the drains: 1 m
$D = 3$ m
$q = 0.007$ m/day
$r = 0.10$ or $u = 0.30$

Problem: the calculation of the drain spacing required

Calculation:

$h \quad = 1 - 0.5 = 0.5$ m

Table on graph I	$8h/q = 570$	$4h^2/q = 145$
	$K_2 = 1$	$K_1 = 0.5$
	$8K_2h/q = 570$	$4K_1h^2/q = 72$

Graph I: for $D = 3, L = 35$ m.

The question now arises, if the ground water rises to 0.10 m below ground level as a result of extremely heavy rainfall, how many days will it take to reach again a height of 0.50 m below ground level ($h_o = 0.90$ m, $h_t = 0.50$ m)?
In making this calculation it is first of all necessary for the above drainage situation, in which the radial resistance was also included, to be replaced by a situation in which there is only a horizontal resistance, the reason being that the nonsteady-state formulas take into account only the horizontal resistance.
This can be done in two ways, viz. the D value is reduced to HOOGHOUDT's d value[1]), or the drain spacing L is converted to the higher value of L_o as indicated in Section 6.

Calculations:

$j = VL^2/10\,KD$

for $L = 35$, $D = 3$ m and $r = 0.10$, read from Appendix B: $d = 2.08$.[1])

$$KD = K_2d + K_1D_1 = 1 \times 2.08 + 0.5 \times (0.90 + 0.50) / 4 = 2.26$$

$$V = 0.07; j = (0.07 \times 35^2) / (10 \times 2.26) = 3.8 \text{ days}$$

or, using the note book method ($L \rightarrow L_o$):

$$KD = K_2D_2 + K_1D_1 = 1 \times 3 + 0.5 \times 0.35 = 3.18$$

$$\frac{KD}{K_2} ln \frac{D_o}{u} = \frac{3.18}{1} ln \frac{3}{0.30} = 3.18 \times 2.3 = 7.3$$

$$L_o = 35 + 7.3 = 42.3 \text{ m}$$

$$j = (0.07 \times 42.3^2) / (10 \times 3.18) = 3.9$$

$$h_t/h_o = \frac{0.50}{0.90} = 0.56; \text{ Graph V: } t/j = 0.74 \text{ or } t = 0.74 \times 3.9 = 2.9 \text{ days}$$

[1]) If $D > {}^1/_4\,L$ and d-tables are not available, then d can be calculated by using the formula:

$$d = \frac{0.39\,L}{ln \frac{L}{u}}$$

It will be necessary to ascertain from local experience whether this time required for the ground water to fall to a given level actually meets the crop requirements.

The case in which drains are situated on the barrier layer (Fig. 10).

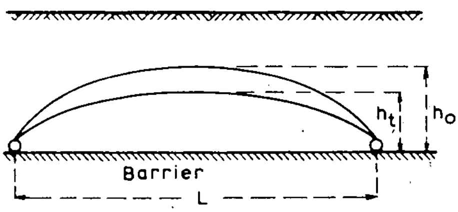


Fig. 10. Falling watertable. Drains placed on the barrier layer

The solution of this case as given by BOUSSINESQ (1903) can be reduced to the following formula, which is only a different way of writing the formula given by DUMM (1954):

$$L^2 = \frac{4.5\, t\, K\, h_o\, h_t}{V\,(h_o - h_t)}$$

(theoretically the constant is 4.46).

No nomograph was constructed for this simple formula as the calculations can be made with sufficient rapidity with a slide-rule.

APPENDIX A

CALCULATION OF HOOGHOUDT'S EQUIVALENT LAYER d

The ground-water flow to the drain can be expressed in the following general form:

$h \; = qL \times R$, in which

$h \; =$ available hydraulic head

$qL =$ rate of ground-water flow from two sides into a unit length of drain

$R \; =$ total resistance = horizontal + radial resistance.

For the radial resistance R_r HOOGHOUDT gives a very long formula in which, however, the first term $1/\pi \, ln \, (0.7 \, D/r)$ predomina testo such an extent that the remainder ot the formula may be omitted without impairing the required accuracy. This is also shown by the calculation of the equivalent layer d.

The horizontal resistance R_h is calculated by HOOGHOUDT up to a distance of 0.7 D from the drains. Hence this becomes $(L - 1.4 \, D)^2 \, / \, 8 \, DL$ and $d = L \, / \, 8 \, (R_h + R_r)$.

When, for example, $L = 30$ m, $D = 5$ m and $r = 0.10$ m, then:

$$R_h = \frac{(L - 1.4D)^2}{8DL} = \frac{23^2}{8 \times 5 \times 30} = \frac{529}{1200} = 0.44$$

$$R_r = \frac{1}{\pi} \, ln \frac{0.7D}{r} = \frac{1}{3.14} \, ln \frac{3.5}{1.10} = 1.13$$

$$d = \frac{L}{8 \, (R_h + R_r)} = \frac{30}{8 \, (1.13 + 0.44)} = \frac{30}{12.56} = 2.39$$

According to the d-tables of HOOGHOUDT (Appendix B), when $L = 30$, $D = 5$ and $r = 10$, $d = 2.38$, which is in close agreement with the d-value calculated.

The above can be represented by the following general formula:

$$R_h \, (D, L - 1.4 \, D) + R_r \, (0.7 \, D) = R_h \, (d, L)$$

$$0.44 + 1.13 = L/8d = 30/8 \times 2.39$$

$$1.57 = 1.57$$

APPENDIX B

d-VALUES HOOGHOUDT (1940)

$r = 0.10\ m$

D / $L \rightarrow$	5 m	7,5	10	15	20	25	30	35	40	45	50
0.5 m	0,47	0.48	0.49	0.49	0.49	0.50	0.50				→
0.75	0.60	0.65	0.69	0.71	0.73	0.74	0.75	0.75	0.75	0.76	0.76
1.00	0.67	0.75	0.80	0.86	0.89	0.91	0.93	0.94	0.96	0.96	0.96
1.25	0.70	0.82	0.89	1.00	1.05	1.09	1.12	1.13	1.14	1.14	1.15
1.50		0.88	0.97	1.11	1.19	1.25	1.28	1.31	1.34	1.35	1.36
1.75		0.91	1.02	1.20	1.30	1.39	1.45	1.49	1.52	1.55	1.57
2.00			1.08	1.28	1.41	1.5	1.57	1.62	1.66	1.70	1.72
2.25			1.13	1.34	1.50	1.69	1.69	1.76	1.81	1.84	1.86
2.50				1.38	1.57	1.69	1.79	1.87	1.94	1.99	2.02
2.75				1.42	1.63	1.76	1.88	1.98	2.05	2.12	2.18
3.00				1.45	1.67	1.83	1.97	2.08	2.16	2.23	2.29
3.25				1.48	1.71	1.88	2.04	2.16	2.26	2.35	2.42
3.50				1.50	1.75	1.93	2.11	2.24	2.35	2.45	2.54
3.75				1.52	1.78	1.97	2.17	2.31	2.44	2.54	2.64
4.00					1.81	2.02	2.22	2.37	2.51	2.62	2.71
4.50					1.85	2.08	2.31	2.50	2.63	2.76	2.87
5.00					1.88	2.15	2.38	2.58	2.75	2.89	3.02
5.50						2.20	2.43	2.65	2.84	3.00	3.15
6.00							2.48	2.70	2.92	3.09	3.26
7.00							2.54	2.81	3.03	3.24	3.43
8.00							2.57	2.85	3.13	3.35	3.56
9.00								2.89	3.18	3.43	3.66
10.00	↓	↓	↓	↓	↓	↓	↓	↓	3.23	3.48	3.74
∞	0.71	0.93	1.14	1.53	1.89	2.24	2.58	2.91	3.24	3.56	3.88

D / L →	50	75	100	150	200	250
0.5 m	0.50					→
1	0.96	0.97	0.98	0.99	0.99	0.99
2	1.72	1.80	1.85	1.90	1.92	1.94
3	2.29	2.49	2.60	2.72	2.79	2.83
4	2.71	3.04	3.24	3.46	3.58	3.66
5	3.02	3.49	3.78	4.12	4.31	4.43
6	3.23	3.85	4.23	4.70	4.97	5.15
7	3.43	4.14	4.62	5.22	5.57	5.81
8	3.56	4.38	4.95	5.68	6.13	6.43
9	3.66	4.57	5.23	6.09	6.63	7.00
10	3.74	4.74	5.47	6.45	7.09	7.53
12.5		5.02	5.92	7.20	8.06	8.68
15		5.20	6.25	7.77	8.84	9.64
17.5		5.30	6.44	8.20	9.47	10.4
20			6.60	8.54	9.97	11.1
25			6.79	8.99	10.7	12.1
30				9.27	11.3	12.9
35				9.44	11.6	13.4
40					11.8	13.8
45					12.0	13.8
50					12.1	14.3
60	↓	↓	↓	↓	↓	14.6
∞	3.88	5.38	6.82	9.55	12.2	14.7

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LIST OF NOTATIONS

a b	=	dimensionless parameters
d	=	thickness of the 'equivalent layer', introduced by HOOGHOUDT in this formula as to take into account the radial flow in the vicinity of the drains (metres)
D	=	in HOOGHOUDT's formula: mean depth of the impermeable layer below the drains (m) in ERNST's formula: thickness of the layer(s) for the horizontal flow component, to be specified in further detail according to the hydrological situation (D_1, D_2)
D_1	=	average cross section for the horizontal component in a layer of permeability K_1
D_2	=	thickness of the second layer for the horizontal component
D_o	=	thickness of the layer for the radial flow component (ERNST)
D_v	=	thickness of the layer for the vertical flow component (ERNST)
e	=	base of the natural logarithm $ln = 2.72$
h	=	height of the water table above drain level midway between the drains (metres)
h_o	=	midpoint water table height at beginning of drainout period
h_t	=	midpoint water table height at end of drainout period
j	=	reservoir-coefficient (in *days*). In the non-steady flow formula it characterises the drainage situation by incorporating the main hydrological properties of the situation
K	=	hydraulic conductivity (m/day)
L	=	spacing of drains measured between centres (metres)
ln	=	$\log_e$ ..... = 2.3 $\log_{10}$ .....
q	=	rate of rainfall or drain discharge per m² of area drained (m³/m²/day /= m/day)
r	=	radius of drain (m)
R_v	=	Resistance for the vertical flow component
R_h	=	Resistance for the horizontal flow component
R_r	=	Resistance for the radial flow component
t	=	time
u	=	wetted perimeter of drain (metres)
V	=	volume-fraction of pores drained at a falling water table

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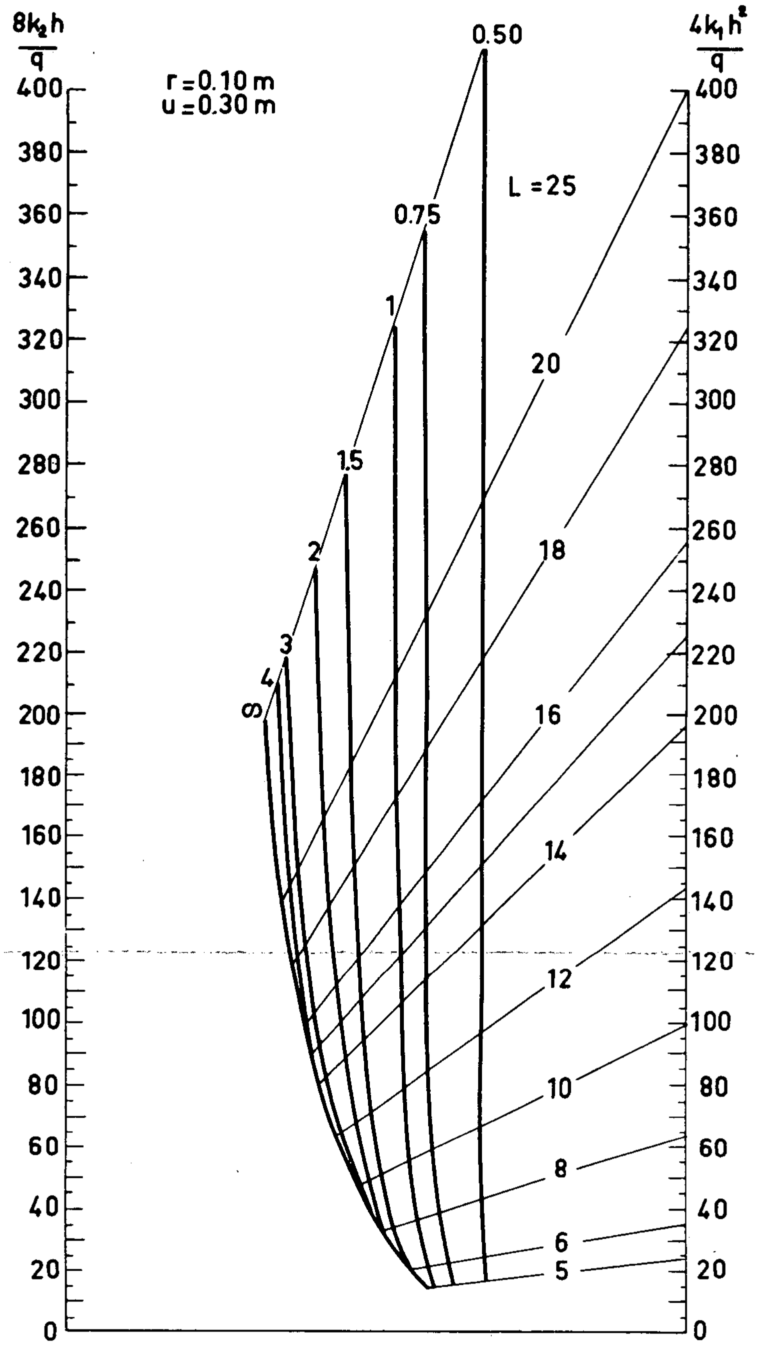
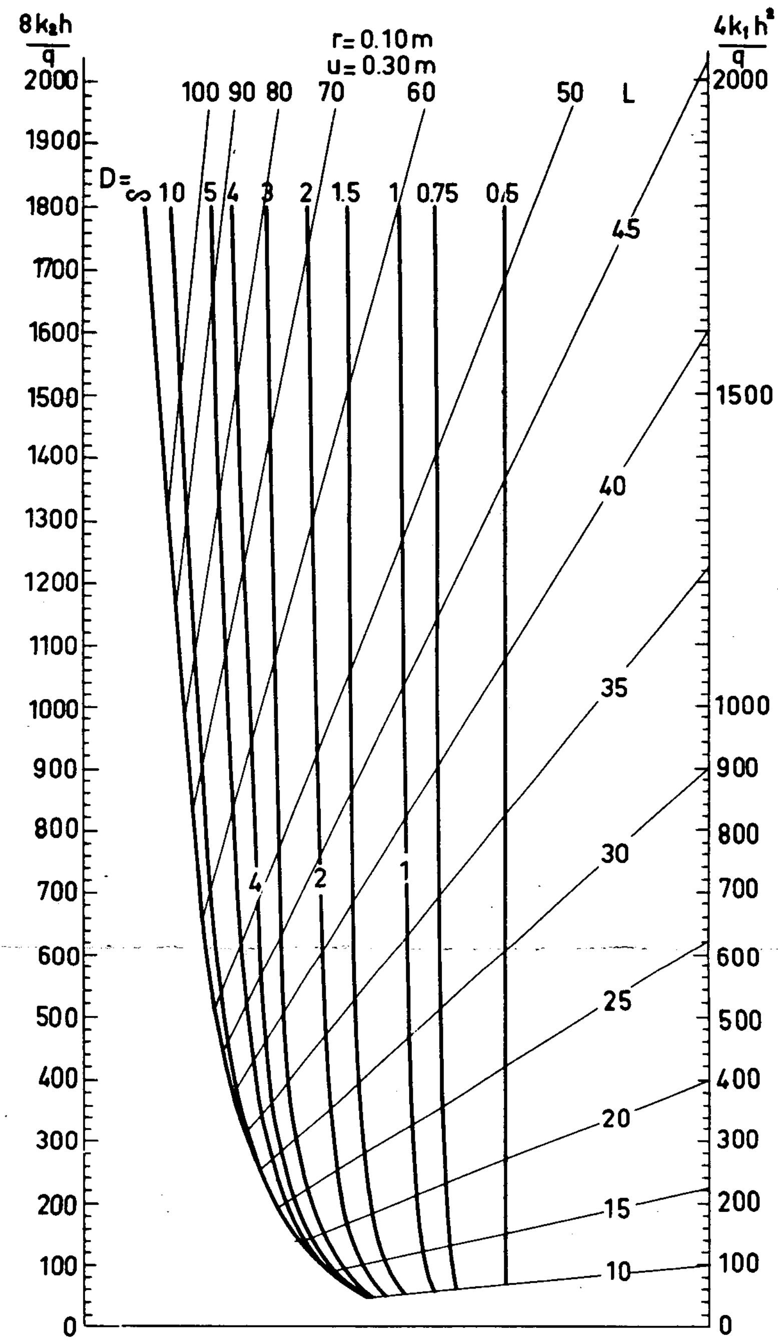
Graph I TILE DRAINS (formula Hooghoudt)

Graph Ia: $L = 5 — 25$ m.



Graph Ib: $L = 10 — 100$ m.



$\frac{8h}{q}$ (first number) and $\frac{4h^2}{q}$ (second number) for various h- and q-values

	q in mm per day									
h (meters)	1	2	3	4	5	6	7	8	9	10
0.1	800– 40	400– 20	265– 15	200– 10	160– 8	135– 10	115– 5	100– 5	90– 5	80– 5
0.2	1600– 160	800– 80	530– 55	400– 40	320– 32	265– 30	230– 25	200– 20	180– 20	160– 15
0.3	2400– 360	1200– 180	800– 120	600– 90	480– 70	400– 60	345– 50	300– 45	270– 40	240– 35
0.4	3200– 640	1600– 320	1070– 215	800– 160	640– 130	530–110	455– 90	400– 80	360– 70	320– 65
0.5	4000–1000	2000– 500	1340– 335	1000– 250	800– 200	665–165	570–145	500–125	445–110	400–100
0.6	4800–1440	2400– 720	1600– 480	1200– 360	960– 290	800–240	685–205	600–180	535–160	480–145
0.7	5600–1960	2800– 980	1860– 650	1400– 490	1020– 390	930–325	800–280	700–245	620–215	560–195
0.8	6400–2560	3200–1280	2140– 850	1600– 640	1280– 510	1070–425	915–365	800–320	710–285	640–255
0.9	7200–3240	3600–1620	2400–1080	1800– 810	1440– 630	1200–540	1030–460	900–405	800–370	720–325
1.0	8000–4000	4000–2000	2700–1330	2000–1000	1600– 800	1330–665	1140–570	1000–500	890–445	800–400
1.1	8800–4840	4400–2420	2940–1600	2200–1210	1760– 970	1460–805	1260–690	1100–605	980–535	880–485
1.2	9600–5760	4800–2880	3200–1920	2400–1440	1920–1150	1600–960	1370–820	1200–720	1060–640	960–575

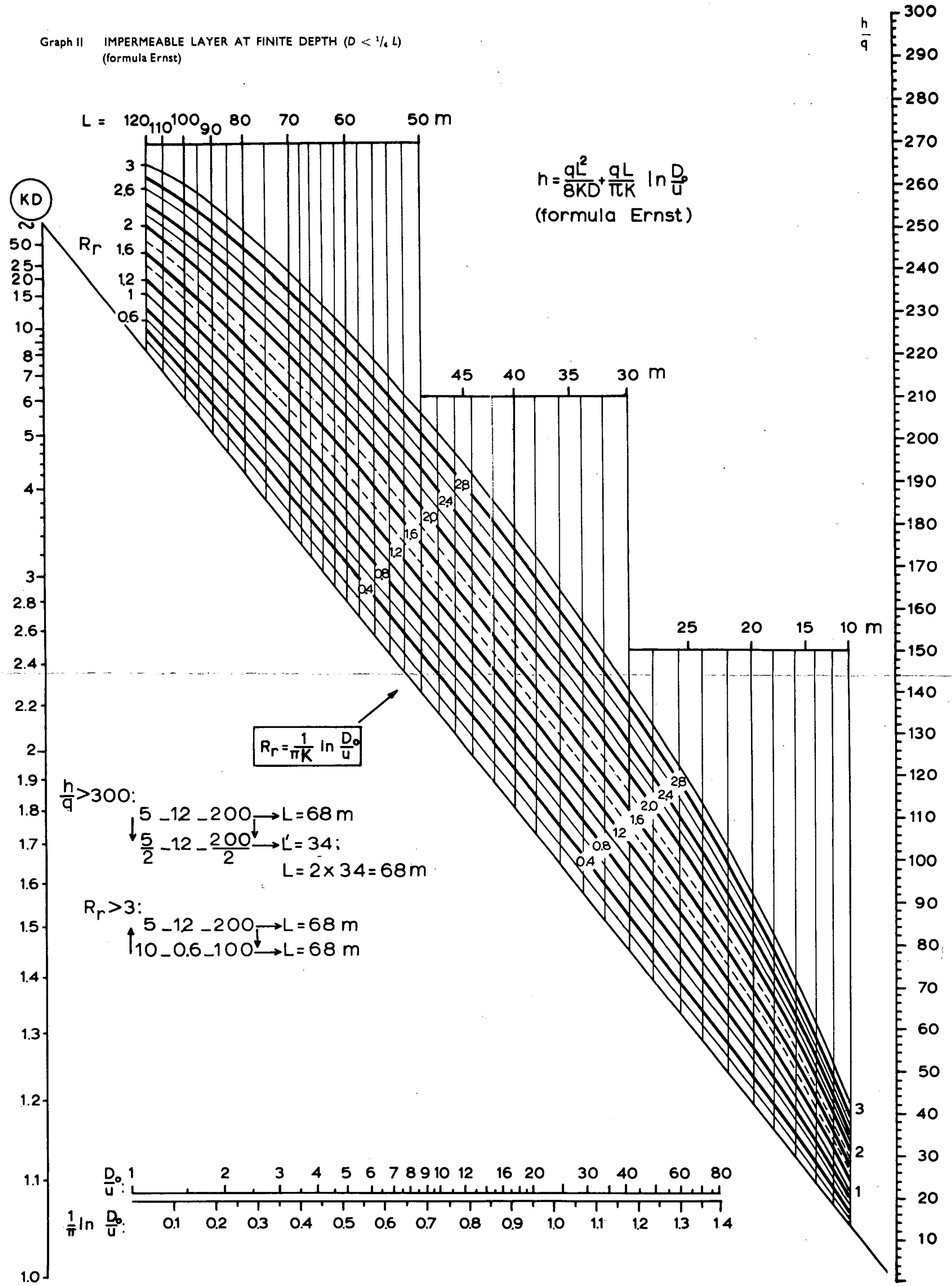
Example: $h = 0.5\ m$, $q = 7\ mm$ per day: $\frac{8h}{q} = 570$; $\frac{4h^2}{q} = 145$

GRAPH I

Tile drains

(formula Hooghoudt)

Graph II IMPERMEABLE LAYER AT FINITE DEPTH ($D < {}^1/_4 L$)
(formula Ernst)



GRAPH II

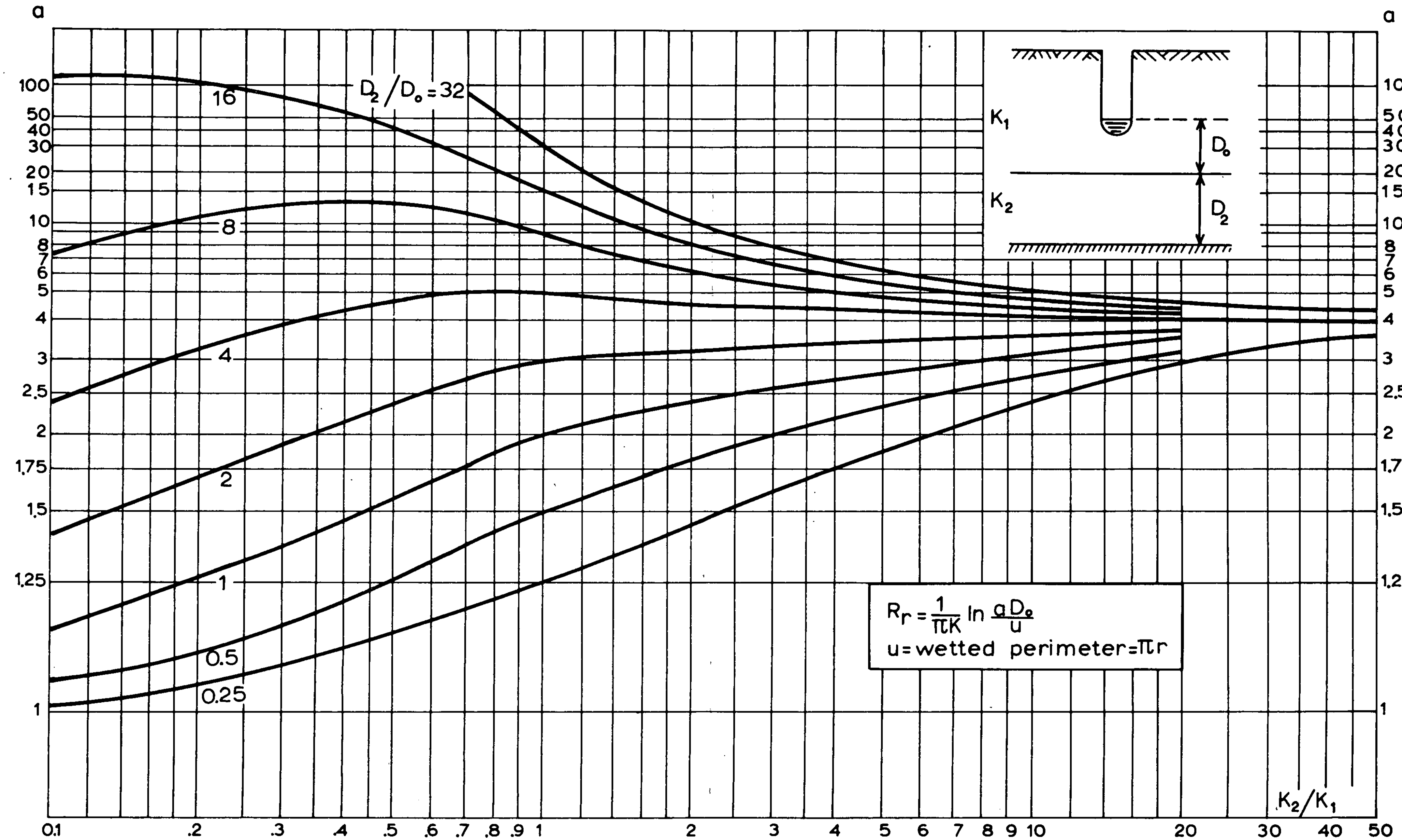
Impermeable layer at finite depth

($D < {}^1/_4\ L$)

(formula Ernst)

Graph IIa RADIAL RESISTANCE (R_r) – SOIL BELOW THE DRAINS CONSISTS OF TWO DIFFERENT LAYERS

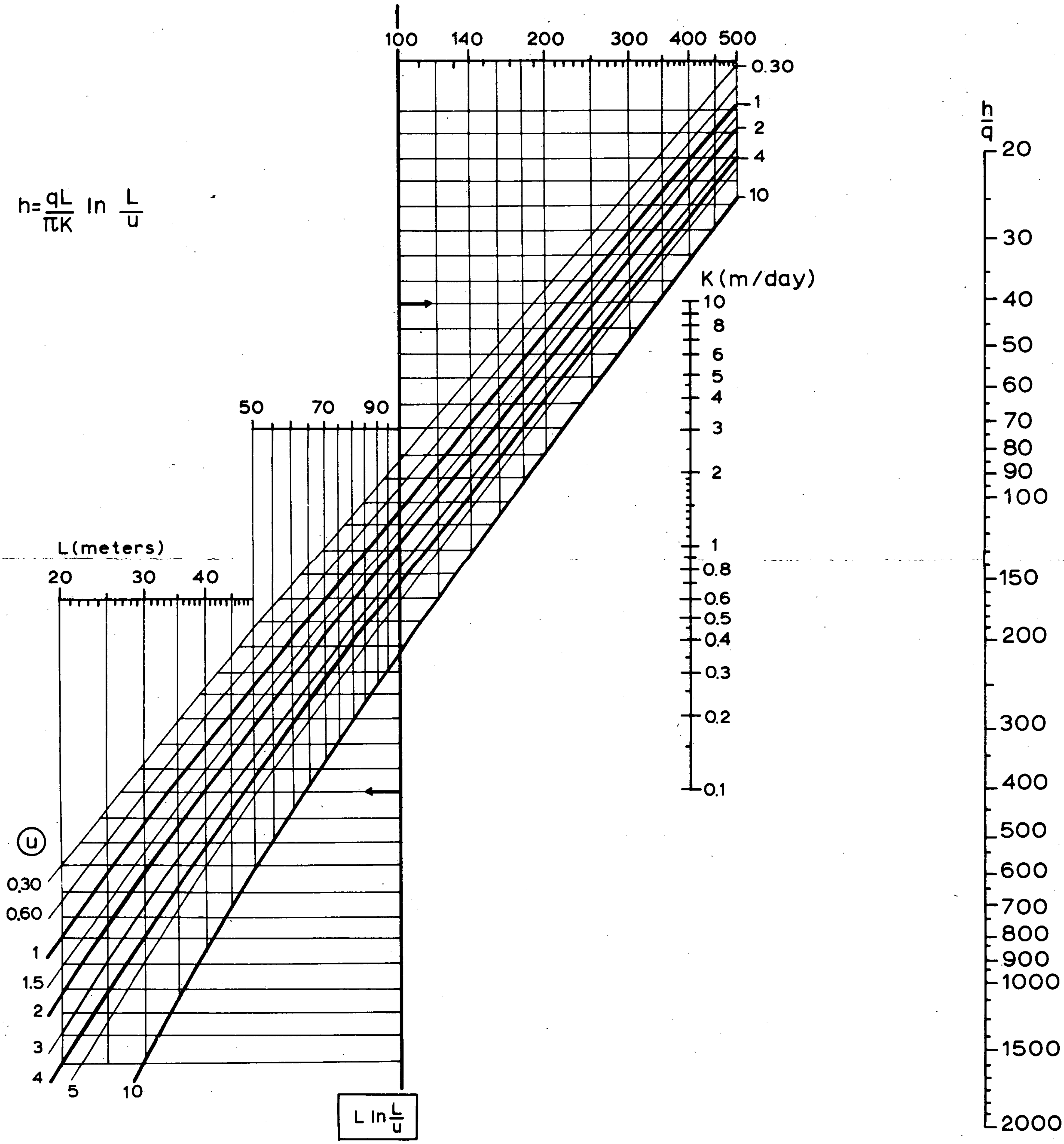
(formula Ernst)



GRAPH IIa

Radial resistance (R_r)

Graph III IMPERMEABLE LAYER AT A GREAT DEPTH ($D > \frac{1}{4} L$)



GRAPH III

Impermeable layer at a great depth

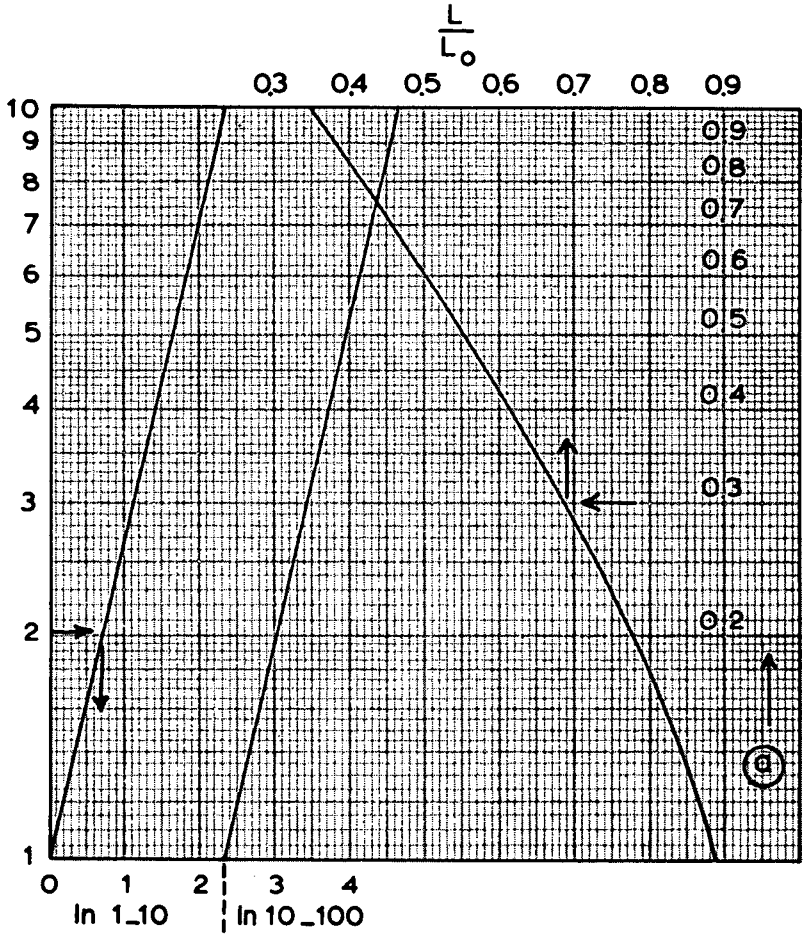
($D > {}^1/_4\, L$)

Graph IV NOTEBOOK METHOD

$$L_o^2 = \frac{8KDh}{q}$$

For homogeneous soil and $D \ln \frac{D_o}{u} < 0.4\ L_o$:

$$L = L_o - D \ln \frac{D_o}{u}$$



example: $\ln 2 = 0.7$

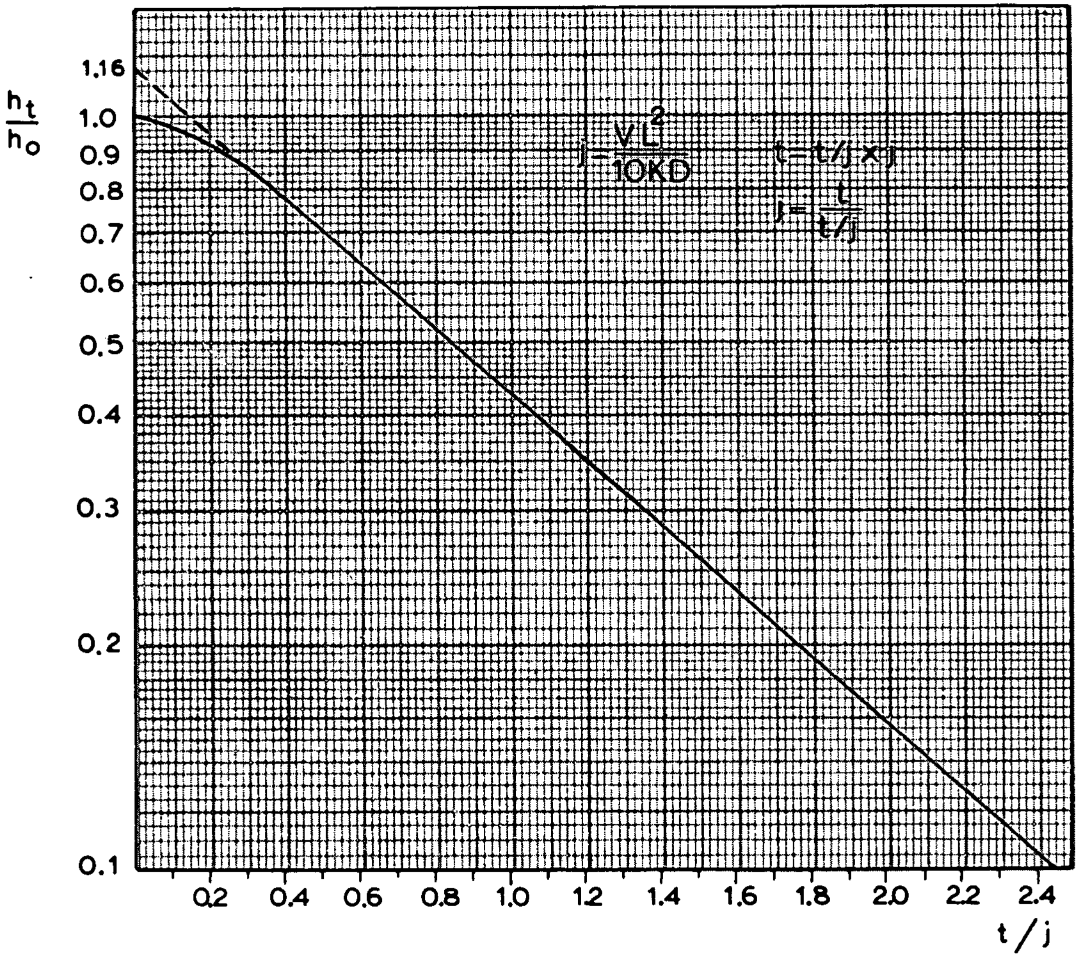
$\ln 20 = 3.0$

$a = 0.3 \longrightarrow \frac{L}{L_o} = 0.69$

$$a = \frac{KD}{K_2 L_o} \ln \frac{D_o}{u}$$

or for homogeneous soil

$$a = \frac{D}{L_o} \ln \frac{D_o}{u}$$


$\frac{h_t}{h_o}$
1.16
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
$j = \frac{V L^2}{10 K D}$
$t = t/j \times j$
$j = \frac{t}{t/j}$
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
t / j