

DRAINAGE PRINCIPLES AND APPLICATIONS

III SURVEYS AND INVESTIGATIONS

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DRAINAGE PRINCIPLES AND APPLICATIONS

- I INTRODUCTORY SUBJECTS
- II THEORIES OF FIELD DRAINAGE AND WATERSHED RUNOFF
- III SURVEYS AND INVESTIGATIONS
- IV DESIGN AND MANAGEMENT OF DRAINAGE SYSTEMS

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International Course on Land Drainage
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PREFACE (to Volume III)

The International Institute for Land Reclamation and Improvement has been organizing the International Course on Land Drainage in Wageningen, The Netherlands, each year since 1962. In 1969 the Board of the course decided to have the entire lecture notes re-edited and issued by the Institute in a simple four-volume publication. Readers interested in the reasons for this decision are referred to the Preface and Introduction of Volume I, which was issued in 1972, followed by Volume II in 1973. These two volumes deal with introductory subjects and the theories of field drainage and watershed runoff.

This book is the third volume in the series and describes the various surveys and investigations required before an artificial field drainage system can be planned and designed.

After an introductory chapter on surveys and their sequence (Chapter 17) currently applied methods for the analysis of rainfall data are treated in Chapter 18 and the methods for determining evapotranspiration in Chapter 19.

In general, common soil surveys do not provide an adequate factual basis for drainage designs. An additional or special survey is usually necessary to determine such hydrological soil properties as infiltration and percolation rates, the storage of water in the soil, and the movement of the groundwater through the soil layers. These aspects are presented in Chapter 20. Chapter 21 describes the basic elements of a groundwater survey for drainage purposes. The assessment of a groundwater balance, presented in Chapter 22, can be regarded as a means of determining the actual cause of the drainage problem.

Determining the physical and hydrological properties of a soil (soil moisture tension, soil moisture content, hydraulic conductivity, transmissivity of aquifers, hydraulic resistance of confining layers, and effective porosity) are important subjects in nearly

all land drainage studies. Field and laboratory methods for determining these characteristics are discussed in the remaining Chapters 23 through 26.

Although each volume can be used separately reference is often made to the other volumes to avoid repetition. The four volumes complement one another and it is hoped that they will provide a coverage of all the various topics useful to those engaged in drainage engineering.

Most of the chapters of this volume underwent minor or major editorial changes, some chapters even being completely revised or rewritten because another lecturer had taken over the subject (Chapter 19) or because the subject matter had to be updated (Chapter 20).

The members of the Working Group who contributed to the editing of Volume III were:

Mr. N. A. de Ridder, Chairman, Editor-in-Chief

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I would like to express my thanks to everyone involved – the members of the Working Group, authors, lecturers, and draughtsmen – for their combined efforts in achieving this result. May this volume be received with the same interest as the previous ones.

Wageningen, March 1974

F. E. Schulze

Director

International Institute for

Land Reclamation and Improvement

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2	Soils and soil properties	W. F. J. VAN BEERS
3	Salty soils	B. VERHOEVEN
4	Plant growth in relation to drainage	G. A. W. VAN DE GOOR
5	Physics of soil moisture	P. H. GROENEVELT
		J. W. KIJNE
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		W. H. VAN DER MOLEN

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15	Rainfall-runoff relations and computational models	D. A. KRAIJENHOFF
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SURVEYS AND INVESTIGATIONS

17. SURVEYS AND THEIR SEQUENCE

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PURPOSE AND SCOPE

Some observations on the phasing of project surveys, with particular reference to drainage.

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17.1 OBJECTIVES AND PHASING OF PROJECT SURVEYS

The cases will be rare where drainage would be the only and exclusive operation to achieve a project's purposes. Even in temperate humid regions a land improvement or reclamation project usually implies a number of combined actions involving water management, infrastructure, land consolidation, agricultural extension, etc. In arid regions the emphasis will be on the water supply by irrigation, while the drainage system - although in many cases an essential and integral part of such projects - is a secondary item in the whole. Elsewhere one may find extensive project areas where a combination of measures is needed in varying intensity.

It may occur as well that, in a valley where the original problem was seasonal waterlogging, the groundwater table is considerably lowered after the introduction of irrigation by pumping from wells, so that finally drainage provisions can be disregarded.

These few examples indicate that drainage works have varying degrees of importance in a land development plan but seldom give rise to a project on their own. It therefore has limited sense to speak about "drainage projects" as such.

Much of the information needed for considering drainage works is the same as that needed for any land development project, both requiring geological, topographic, and soil maps, climatic and agricultural data, and so on. However, the introduction of drainage involves some specific studies and surveys. It will, for instance, call for special analyses of climatic data and groundwater balances; it will require detailed information on hydraulic conductivity and moisture characteristics of soils; and it presupposes knowledge about the practical application of various possible drainage systems. Other chapters in these volumes deal with those questions in detail; the present chapter aims only to introduce some organizational principles concerning the surveys needed.

To arrive at a final answer whether or not a project proposal is technically feasible and economically sound, a sequence of studies of increasing intensity will usually be required. Each phase in itself will involve successive actions: the collection of data, their analysis, the formulation and design of one or more project proposals, and their evaluation.

As a general rule three phases of study are to be recommended:

- at reconnaissance level
- at semi-detailed level
- at detailed level.

Such an approach is a safeguard against spending time, energy, and costs on studies that might afterwards prove to have been superfluous. The whole of this sequence of studies should start with a clear statement of the objectives one is aiming at, giving due regard to the country's development objectives and immediate priorities; this should form the basis for programming the studies at the reconnaissance level. If the outcome of the first phase is favourable, it should be followed by recommendations for the programme of the following phase of study, or finally for implementation. The programme at each level should specify the time, manpower, equipment, and costs involved. If the conclusion at any phase is negative, due to insurmountable technical or economic problems, the studies will be finished.

The recommended phasing may be slightly different in each individual case depending on the nature and extent of the project area. In large areas and projects of a complex nature, more phases may be needed to allow frequent consultations with all interested organizations; after the definition of a master plan for the whole, the area may be split up into smaller units that are studied separately and executed in a certain order of priority. This procedure may also be indicated from a viewpoint of spreading investments and continuity of work for the agencies concerned.

On the other hand, in regions or countries where much information is available on the conditions in the area, or where much experience exists with the criteria of the particular improvement measures, the three phases may be reduced to two, or in the case of small areas, even to one.

The fact that the costs of these types of studies are not negligible stresses the usefulness of investigating in stages; it moreover pleads for a relatively not too small project area so as to maintain a reasonable rate for the costs of studies per unit area. This is of particular importance for projects of a complex nature where an expert team of various disciplines has to be recruited, especially if they are to come from outside the country. From this viewpoint a 10,000 hectares area might be considered a rather small project. As a rough guideline, the order of magnitude of the costs for land development studies may be 5 to 10 percent of the actual project costs; this rate covers the three levels of study together, but excludes the costs of control and supervision of the construction, and disregards the normal operating costs of the associated government agencies. The same argument in favour of a minimum project size is true if foreign loans are to be attracted for the implementation of the project (from the World Bank for instance); for a small area the costs of guidance and control would be relatively too heavy.

17.2 THE MAIN STUDY PHASES

Without claiming completeness the following paragraphs are meant to elaborate the purpose and the nature of actions for each of the recommended study phases, special regard being given to their drainage aspects.

17.2.1 RECONNAISSANCE SURVEY

The main objective of a reconnaissance survey is to identify the feasibility of the proposed project, first of all on technical, but also on economic grounds. This survey should give an answer to the following questions:

- What area or areas can be considered for improvement?
- What advantages and disadvantages can be expected from the intended change of the existing state?
- What are the alternative possibilities for improvement?
- What technical and organizational measures have to be taken for the alternative solutions?
- What will roughly be the proportion of costs to benefits in each case?

In most cases only part of the advantages and disadvantages can be expressed in terms of money-value. There are often important imponderable factors which should also be taken into account, e.g. the creation of new employment possibilities, the better accessibility of the area or the development of shorter transport routes, the creation or destruction of attractive landscapes, recreational facilities or cultural values; the improvement of public health by the elimination of breeding grounds of diseases (e.g. malaria and bilharzia by the reclamation of marshes), etc. Hence the proportion of the costs to the benefits in this stage is only one of the factors to be considered in taking a decision for or against moving to the second stage of investigation: the semi-detailed survey.

A study at reconnaissance level will be mainly based on existing information but includes some limited field work. Documents concerning the area under consideration should be collected from all agencies which ever worked in the area. Very welcome are earlier studies, which should be analysed and re-evaluated from the present-day viewpoint. Other pertinent data are:

- aerial photographs
- all kinds of maps: geological, topographic (scale preferably not smaller than 1/25,000 or 1/50,000), elevation maps of the land surface, road maps, land utilization and ownership maps, etc.
- existing data on soils, surface water, groundwater, climate, crops, crop yields, etc.

The maps should cover the whole of the watershed basin in which the area under consideration is located. These will serve as the basis for water regime studies and water balance computations. If the area covers only a small part of the basin the data should allow estimates to be made of the hydrologic, and possibly other, consequences of mutual interaction with the other part of the basin.

The field work in this stage is mainly to enable the investigator to familiarize himself with the general conditions and to collect some complementary information through inquiries or through incidental observations.

One of the first things to be done is a provisional delimitation of the area to be identified as a project. For large areas sub-units may be identified and a priority order defined for further studies.

With respect to the drainage features the reconnaissance survey should determine the extent of excess water occurrence, trace all various causes of this excess, and try to estimate quantitatively their inputs, frequency, and duration. In this connection it may be useful to outline the extent of inundated spots as they have occurred over a series of years, for instance through inquiring of local inhabitants. Such data can possibly be related to precipitation or level and discharge data of near-by rivers. In case of inundations obviously flood protection or coastal embankment may be more essential than a drainage system, although such measures are often complementary.

Any indications about depth-to-groundwater-table in relation to precipitation and near-by surface water levels will be invaluable. If no systematic water table observations are yet available, inquiries directed to local inhabitants who are using wells may provide useful information.

For low-lying areas and natural depressions an obstacle to the discharge of excess water may be the location and condition of the natural "outlet". This item may need particular attention. For inland areas solutions for gravity discharge are sometimes found by blowing up a threshold in a river bed or constructing a tunnel to a near-by lower outlet.

For low-lying coastal land one may take advantage of the tidal effect by means of tide locks, which discharge during low tide and close automatically when the outside level rises.

Where the gravity outlet does not offer good prospects, discharge by pumping may be a possible alternative; a pump installation moreover can advantageously be installed at a different place than the natural outlet. Being rather decisive for

the project, such a solution should be carefully studied at the reconnaissance level; consideration should be given not only to the installation of the pump but also to the operational costs, which will require estimates of the volume of water to be lifted annually and the maximum per day, height of the lift, expenditure on energy and personnel, etc. Criteria for the water control system and its management should be established for the proposed cultivation programmes, taking soil conditions into account, after which one or more project designs can be developed. Due attention should also be given in this stage to possible constraints with regard to water levels, water quality, discharge, or otherwise, which might limit the choice of solutions or would require special provisions. Some examples are:

- built-up areas and road systems may demand extra deep drainage; in contrast wooden foundations may need a shallow groundwater level to prevent them from rotting;

- if polluted water from urban areas is discharged into surface water, provisions might have to be made to safeguard the interests of public health, fisheries, wildlife, and recreation;

- lowering the phreatic level in the area under consideration by drainage might adversely affect the hydrologic circumstances of neighbouring areas;

- parts of the area may be situated in different administrative units or different water districts, and may not be subject to the same regulations.

The reconnaissance survey should devote a substantial part of its investigations to the collection of hydrologic data on the existing water regime, as this will provide the basis for future water management.

Hydrologic data, due to their natural variations, will usually be expressed in terms of probability per time unit and should therefore cover a reasonably long period to be of any value; a ten years range might be considered a strict minimum but much depends on their relative variability and the purpose of use. Correlations can be sought between available climatic data and other data that may be scarce: for instance between precipitation and river discharges or between temperature and evaporation. In this manner one may arrive at an estimated longer range of data. If, however, sufficiently reliable data are not available to allow an estimate of the water balance components, an immediate programme of appropriate observations will have to be set up.

Similarly an early verification of the response of proposed crops to the assumed water management conditions will be needed. If no local experience is available

and no comparison with near-by areas is possible, one might seriously consider the lay-out of experimental fields for this purpose. Of course, such field experiments cannot be completed during the phase of the reconnaissance survey. Hence conducting such experiments should only be considered seriously if, on other grounds, there is sufficient evidence for the technical feasibility of the project. Field experiments, if conducted for a number of consecutive years, will be of invaluable help in studying crop response under controlled conditions. A supplementary advantage of such experimental fields, if used for longer periods, is that they offer the prerequisites for a demonstration and education centre for the farmers in the area. The extent and number of the experimental fields should be chosen in proportion to the size of the project area and the variety of problems to be studied: sometimes it is worth-while to include trials with different types of field drainage systems.

The study at reconnaissance level should conclude with a report which summarizes all existing knowledge and formulates possible alternative solutions to the problem. Accompanying the report should be maps showing the borders and sketch-plans of the area, including the approximate location of the main elements of the water management system.

Costs and production figures needed for the preliminary appraisal at this stage can be derived from experience elsewhere (expressed in units of area or length), if possible adapted to the prevailing local conditions. The most important items in the report are the recommendations on the next steps to be taken; if one or more of the proposed alternative solutions are considered to be feasible, it should be indicated whether or not additional investigations at reconnaissance level are needed and/or what programmes of surveys and studies are needed in the semi-detailed phase.

17.2.2 SEMI-DETAILED SURVEY

This study comprises the additional activities needed to work out the alternative sketch plans retained from the reconnaissance study up to a "semi-detailed" level (also indicated as "preliminary plans"). The main difference between this and the earlier stage is that more detail is needed, for which field surveys (defined in the reconnaissance report) will have to be conducted.

The data to be collected should be sufficiently detailed to permit a design of the project works, the costs of which can be estimated to an accuracy of some 10 per cent. At this level the costs and benefits are determined on the basis of calculated quantities and locally checked prices.

The topographic and soil maps required at this stage are usually at scales 1/25,000 or 1/10,000; contour lines of 0.25 m maximum interval will be needed. At the location of projected canals, ditches, and structures, the necessary detailed levelling for length and cross-sections should be performed, and all further surveys needed for the design should be completed. A sample survey outline is found in National Engineering Handbook, Section 16 (see list of literature).

These semi-detailed studies correspond to the level of what is frequently called "feasibility studies". This term, introduced by the International Bank for Reconstruction and Development (IBRD) and associated bodies, is nowadays widely used for studies where the Bank is called upon to assist in financing. Guidelines have been prepared by the Bank for various types of projects, including one for irrigation and drainage. To illustrate the approach they take in judging the viability of project proposals, the Introduction of the latter document is annexed to this chapter. For readers interested in more details of a semi-detailed study report, no better reference could be given at the moment than the above-mentioned "guideline" prepared by FAO/IBRD (see list of literature).

On the basis of the results of the semi-detailed study, the competent authority should finally select one of the plans and decide on execution. But here again, it should be mentioned that the final decision cannot be taken without attention being paid to the imponderable advantages and disadvantages of the undertaking.

17.2.3 DETAILED SURVEY

The best plan for the area under consideration is chosen after conclusion of the semi-detailed studies. What remains to be done, if one has decided on implementation, is the final revision and in particular the elaboration - through calculation and drawing - of the details of structures (bridges, culverts, pumping stations, etc.) to complete the "definitive design".

Estimates for construction costs in this stage will not differ greatly from those estimates made at the semi-detailed level.

The maps now required are at scale 1/10,000 to 1/2,500: structures will require even larger scales.

The final step is to prepare specifications to be put out to tender.

ANNEX

INTRODUCTION¹

As for all types of project, the objective of a feasibility study for an irrigation or drainage project is to demonstrate that the project is:

- in conformity with the country's development objectives and immediate priorities;
- technically sound, and the best of the available alternatives under existing technical and other constraints;
- administratively workable;
- economically and financially viable.

In the context of this paper, a feasibility study is a comprehensive document which will provide all the answers to questions on the above points which might be put by an appraisal team of the World Bank Group.

In formulating (designing) a project there should be a constant effort to minimize costs (but not at the expense of safety), maximize returns, and bring about utilization of the investment in the quickest possible time. The latter will usually necessitate a rapid transformation of the farming practices in the project area. From these considerations stem the main themes of an irrigation project feasibility study:

- A thorough study of the physical resource base, particularly the project area soils, climate, and water supply, in order to ensure that the cropping patterns proposed and the yields predicted can be maintained for a sustained period and in order to determine the scale of the project.
- A thorough examination of the people likely to be involved in the project in order to ensure that the proposed development is appropriate to their attitudes and capacities.
- A thorough study of the engineering alternatives for serving and draining the project lands, and their phasing, in order to ensure that the most appropriate economical but safe solution is achieved.
- An adequate preliminary design of, and a construction schedule for, the works, both project works and on-farm works, in order to demonstrate their suitability and to estimate their costs and the phasing of those costs.

¹ Introduction, copied - after approval from editor - from: FAO/IBRD Coöperative Programme. Guidelines for the preparation of feasibility studies for irrigation and drainage projects. Rome, December 1970. Rotaprint. 25 pp.

- The determination and scheduling of the agricultural pattern (size and type of farm enterprise, crops and their yields) on the basis of physical and human resources, present land use, market projections and prices.

- The determination and phasing of the various measures and inputs necessary to achieve the agricultural plan.

- The determination of the management and organization necessary to construct and implement the project to the time schedules predicted.

- The determination of the economic benefit to the country, the financial returns to the farmers, the financial results of the operating authority, and the repayment of project costs by beneficiaries.

It must be stressed that the main themes of the study are not separate exercises. The finalization of each, and its amalgamation into the whole, is a process of successive approximation reached after cross-consideration of the interim results of the others.

No two projects are the same. There is clearly a wide difference between, for instance, a project using groundwater and sprinklers for the intensive production of vegetables by sophisticated commercial farmers and a project using a simple river diversion for surface irrigation of rice by peasant farmers. These projects are not only different physically, but also in non-physical terms (organization, markets, need for credit, extension effort, etc.). Therefore, any general guideline, such as this paper presents, must be used with intelligence and adjusted to the needs of the particular project under investigation. The guideline is written on the assumption that the main project works will be constructed and owned by a public authority. In the case of groundwater projects, the main works (wells and equipment) may be, and indeed often are, privately owned and financed through a credit operation. For this type of project the forthcoming guidelines to be published on agricultural credit projects should be largely used instead of this guideline.

The guideline does not attempt to deal with special questions of cost allocation which arise in the case of a multiple-purpose project (e.g. when a reservoir is to be used for flood control and/or power in addition to its irrigation purposes). It is assumed in the sections on economic analysis that a cost allocation to irrigation has been made of the joint costs of such a project, but it is recognized that a cost allocation of this nature may itself require a major analytical effort.

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SURVEYS AND INVESTIGATIONS

18. ANALYSING RAINFALL DATA

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PURPOSE AND SCOPE

A description of the statistical techniques for processing rainfall data and a discussion of some probability distributions.

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18.1 INTRODUCTION

For the design of drainage and flood control works, the amounts of water that have to be discharged must be known. If possible, these amounts should be assessed by direct measurements. If not, indirect methods, such as the calculation of discharges from rainfall data, will have to be used. As rainfall is extremely variable in time and space, the rainfall data covering long periods and recorded at various stations will have to be studied. These records can be used in one of two ways:

- All rainfall data, from moment to moment, are fed into a model of the natural system, which has rainfall as input and discharge as output (Chap.15, Vol.II). A design discharge is then selected from the outputs of the model.

- A design rainfall is selected from a range of rainfall values and is then transformed into a design discharge.

This chapter is mainly concerned with the processing of rainfall data and with some statistical techniques that enable a proper design rainfall to be selected. In principle, the same statistical techniques allow a proper design discharge to be selected.

18.1.1 DETERMINING A DESIGN RAINFALL

The amount of rain that falls on the ground in a certain period is expressed as a depth P (mm, inches, etc.) to which it would cover a horizontal plane on the ground. The rainfall depth may be considered a statistical variate, its value depending on

- the season of the year,
- the duration selected,
- the area under study.

In a design, the frequency, season, and duration chosen depend on the type of problem under consideration.

The choice of a design frequency

The higher a rainfall, the less often it occurs. Consequently the higher the design rainfall, implying a more costly project, the less risk there is of failure. There is, however, a certain point at which the cost of ensuring more safety outweighs the benefits of a further reduction in the number of failures. Therefore the choice of a design frequency is an optimization problem.

For large-scale flood protection works, where failure may endanger human life or vital material interests, an average failure of only once in 1000 years or even 10 000 years may be accepted. In this case long-term records have to be available to allow return periods of exceptionally large floods to be predicted with sufficient accuracy by extrapolation (Section 7). If failure is to be understood in an agricultural sense only, i.e. a loss or reduction in agricultural production as may occur in irrigation and drainage projects, an average failure of once in 5 or 10 years is generally accepted. Records covering 20 years may suffice in this case. The statistical techniques to be used may then be relatively simple and restricted to a frequency analysis (Sections 2 and 3).

The critical season

The rainfall analysis for drainage design can be restricted to that part of the hydrological year during which excess rainfall may cause damage (Fig.1). This critical period in The Netherlands is the winter season when rainfall exceeds evaporation. Drainage in winter is necessary to preserve soil structure, to ensure the trafficability and workability of the land, and to enable early seedbed preparation in spring. Drainage in the summer season is of less importance, due to relatively high evaporation rates and consequently good storage facilities for water in the soil.

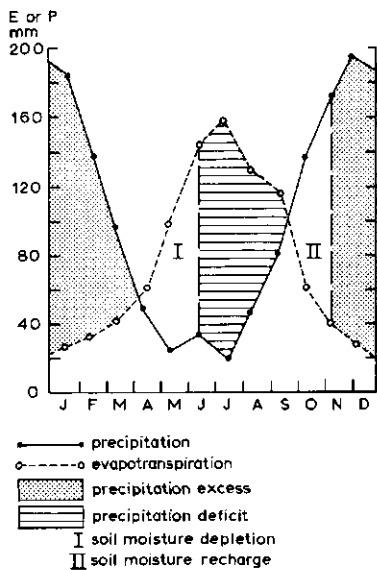


Fig.1. Average monthly precipitation and evapotranspiration (San Francisco, U.S.A.)

Rainfall data

In Roumania, where the monthly rainfall and evaporation pattern is similar to that in The Netherlands, the critical season appears to be the summer season, due to the very high intensity of summer rainstorms, for which the soil does not offer sufficient storage.

When the drainage problem concerns surface drainage for crop protection, it is the growing season that may be critical. If, on the other hand, the problem concerns surface drainage for erosion control, the off-season may be critical because of the erosion hazard on bare soils. Unlike drainage, for which maximum rainfall design values are important, irrigation requires a minimum rainfall design value (maximum evapotranspiration minus effective rainfall). The critical season then is the growing season with a rainfall deficit.

The critical duration

The rainfall intensity is expressed as a depth per unit of time. This unit can be an hour, a day, a month, or a year. The type of problem will decide the duration to be selected for analysis. In a study of the water availability for crop growing or of general water excess, monthly rainfall values may suffice (Fig.2). For irrigation purposes the critical duration depends on the waterholding capacity of the soil and crop response to drought, and is of the order of some weeks. In studies of subsurface drainage, the critical duration depends on the storage capacity of the soil, the design frequency, and the crop response to waterlogging, and is of the order of some days. For main drainage systems the critical duration is often also of the order of some days, depending on the storage allowances of the system and the discharge intensity of the drainage area. For erosion control and the drainage of small, steep watersheds or urban areas, the storage capacities are small; information on hourly rainfalls may then be required.

The analysis of rainfall with respect to duration shows that, with the same periods of recurrence, rainfall intensities decrease as duration increases (Fig.3).

The area under study

Rainfall is measured at certain points. It is likely that the rainfall in the vicinity of a point of measurement is approximately the same everywhere, but farther away from the point this will not be true. It appears that point rainfalls with high return periods are often considerably higher than the area-average rainfall with the same return period (Fig.4). Therefore, the design rainfall for main drainage canals and outlets of large areas can be taken lower than the corresponding point rainfall.

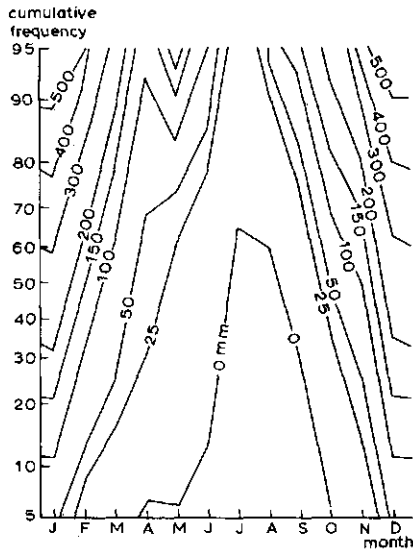


Fig. 2. Frequencies of monthly rainfalls (Antalya, Turkey, 15 years)

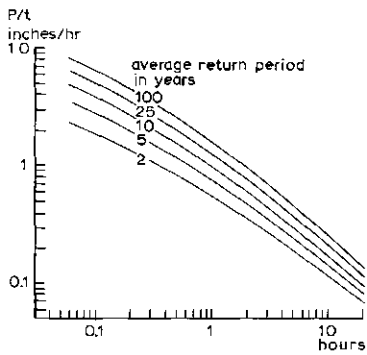


Fig. 3. Depth-duration-frequency relations (Fresno, California, 1903-1941)

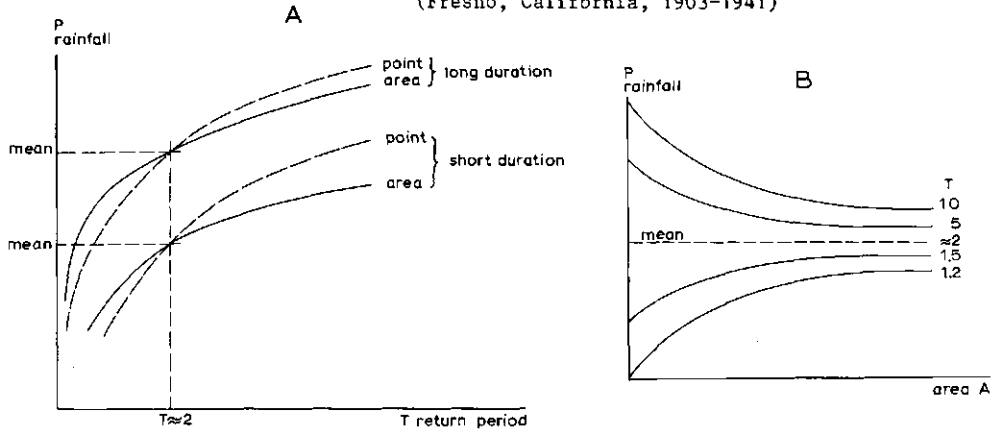


Fig. 4. Schematical presentation of area-depth relations of rainfall. A: Flattening effect of the duration on area-rainfalls as compared with point-rainfalls. B: Flattening effect of size of the area on area-rainfalls of various return periods as compared with point rainfalls

18.2 FREQUENCY ANALYSIS OF RAINFALL AND RECURRENCE PREDICTION

The frequency of rainfall can be analysed directly, as will be discussed in this section, or indirectly through adaptation of data to a probability distribution, as will be discussed in Section 7. The direct frequency analysis is based on the assignment of a frequency to each measured rainfall or to a group of rainfalls in an observation record. Two main procedures of frequency assignment can thus be discerned:

- frequencies based on depth ranking of each measured rainfall; to be used when there are relatively few data,
- frequencies based on depth intervals (grouping of rainfalls); to be used when there are many data.

These procedures will be illustrated below with the data given in Table 1, which presents daily rainfalls for the month of November in 19 consecutive years. This table will also be the basis for all further examples presented in this chapter.

18.2.1 FREQUENCIES BASED ON DEPTH INTERVALS

The procedure is as follows:

- Select an appropriate number (k) of depth intervals (serial number i , lower limit a_i , upper limit b_i) of a width suitable to the data series and the purpose of the analysis,
- Count the number (m_i) of data in each interval,
- Divide m_i by the total number (n) of data in order to obtain the frequency (F) of rainfalls in the i -th interval

$$F(a_i < P < b_i) = \frac{m_i}{n} \quad (1)$$

The frequency thus obtained is called the frequency of occurrence in a certain interval.

How this procedure has been applied for the daily rainfalls given in Table 1 is shown in Table 2, Columns (1), (2), (3), (4), and (5). This last column gives the frequency distribution of the intervals. It can be seen that the bulk of rainfall values was either zero or from 0-1 inches. Greater values, which are of more interest for drainage purposes, are recorded on far fewer days. It appears that the chosen intervals allow a fairly regular grouping of the data.

Table 1. Daily rainfall in inches for the month of November in 19 consecutive years

Date Year	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1948	-	-	-	.12	.10	1.78	.59	-	.04	.21	-	-	.16	.22	-
1949	-	-	.07	.40	.36	.15	.39	-	-	-	-	-	-	-	-
1950	.42	.12	-	.07	.50	-	.33	1.03	.47	.02	.20	.25	-	-	-
1951	1.14	-	.23	3.89	.16	.13	-	-	-	-	-	-	.18	.10	.04
1952	4.37	.32	.32	.84	.03	-	.44	.43	1.01	.02	-	-	.01	-	-
1953	-	-	-	-	.04	.57	.16	1.28	.10	-	.48	-	.45	.60	.10
1954	-	-	-	-	-	-	-	.05	-	.05	.21	.04	.26	-	.02
1955	-	-	-	.40	-	.90	.10	-	1.94	.49	2.25	.06	-	.05	-
1956	-	-	.09	.35	-	-	-	.23	.12	-	-	.09	-	-	.34
1957	.15	-	1.61	1.80	-	-	-	-	-	-	.90	.29	.02	.71	.32
1958	3.64	.13	-	.07	-	-	-	.35	.24	.18	-	.52	-	-	.05
1959	-	2.55	.76	-	1.38	.13	1.05	.38	.01	.50	1.26	.03	.62	.06	-
1960	-	-	-	-	.37	.38	-	-	-	-	-	-	-	-	-
1961	1.61	6.23	.05	-	.40	-	.23	.43	-	-	.03	-	.26	.02	.52
1962	-	.34	-	.38	-	.16	-	-	.06	.18	-	.50	.63	.94	.07
1963	2.93	.26	-	.06	.14	-	.39	-	-	-	-	1.64	.43	-	-
1964	.26	.90	.17	-	.04	.50	1.16	1.57	.51	.56	.05	.15	-	-	-
1965	.44	.50	-	.08	-	-	-	.10	-	.33	2.21	.13	1.73	.21	-
1966	.42	-	-	-	-	.10	-	-	.06	2.12	2.57	.64	-	-	-

	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	Total
1948	-	1.59	.49	-	-	-	-	-	-	.01	-	-	-	-	-	5.31
1949	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1.37
1950	-	-	-	-	-	-	-	.29	.14	-	.85	.20	1.22	2.65	-	8.76
1951	-	-	.02	-	-	-	-	-	-	-	.73	.20	.10	.81	1.81	9.54
1952	.04	.01	-	-	.18	.27	.14	.30	.09	2.10	.13	.23	.05	.13	.83	12.29
1953	.97	.84	-	-	.07	.45	-	.06	.72	1.51	-	.18	.14	.29	.22	9.23
1954	1.42	.14	-	-	-	.15	-	-	.04	-	-	.13	-	-	-	2.51
1955	-	.70	.12	-	-	-	-	.45	.14	-	.02	.02	.91	.58	.07	9.20
1956	.62	.56	.34	-	-	-	-	-	-	-	-	-	-	-	1.17	3.91
1957	.07	.14	-	-	.25	1.48	.13	.54	.07	-	-	-	-	.05	.53	9.06
1958	.66	.85	.12	.03	.77	-	.78	.28	.54	.03	.04	.86	.04	.85	.47	11.50
1959	-	-	-	-	-	-	-	.04	-	-	-	-	.28	.47	-	9.52
1960	-	.27	.11	.12	-	-	-	-	-	1.10	.93	.88	-	-	.32	4.48
1961	.46	.45	-	-	.08	.16	-	-	-	-	-	-	-	-	-	10.93
1962	.26	-	-	-	-	7.93	3.71	.14	-	.21	.02	.46	.57	.01	-	16.57
1963	.05	.15	.02	.54	-	.15	.32	-	-	-	.80	.20	-	1.17	.20	9.45
1964	-	-	.12	.08	-	-	-	-	-	.80	.17	1.46	.60	.24	.17	9.51
1965	-	-	-	.14	.17	.07	.10	-	-	-	.44	.01	-	-	-	6.66
1966	-	.76	.55	-	-	.36	.01	-	-	-	-	-	-	.02	.29	7.90

From the definition of frequency, it follows that the sum of all frequencies equals unity.

$$\sum_{i=1}^k F_i = \sum_{i=1}^k \frac{m_i}{n} = 1 \quad (2)$$

In hydrology one is often interested in the number or frequency of rainfalls exceeding a certain value, viz. the design rainfall. The frequency of exceedance $F(P > a_i)$ of the lower limit a_i of a depth interval i can be obtained by counting the number M_i of all rainfalls exceeding a_i , and dividing this number by the total number of rainfalls (see Table 2, Column 6).

$$F(P > a_i) = \frac{M_i}{n} \quad (3)$$

Frequency distributions are often presented graphically by plotting the frequency of non-exceedance $F(P < x)$ instead of the frequency of occurrence or exceedance. The frequency of non-exceedance $F(P < a_i)$ of the lower limit a_i can be obtained as the sum of the frequencies over the intervals below a_i . The frequency of non-exceedance is also referred to as the cumulative frequency.

From the fact that the sum of all frequencies over all intervals equals unity, it can be derived that

$$F(P > a_i) + F(P < a_i) = 1 \quad (4)$$

The cumulative frequency distribution, therefore, can be directly derived from the distribution of the frequency of exceedance (Table 2, Column 7).

18.2.2 FREQUENCIES BASED ON DEPTH RANKING

The data may be ranked in either ascending or descending order. For a descending order the suggested procedure is:

- Rank the (n) data (P) in a descending order, the highest value first, the lowest last.
- Attach a serial rank number (r) to each value (P_r , $r = 1, 2, \dots, n$), the highest value being P_1 , the lowest P_n .
- Divide the rank number (r) by the total number of observations plus 1 to obtain the frequency of exceedance as:

Table 2. Frequency analysis, based on depth intervals, of daily rainfalls derived from Table 1.

Serial number (1)	Interval (inches)		Number of observations $a_i < P < b_i$ (m_i)	Frequency $F(a_i < P < b_i)$ $\frac{(m_i)}{n}$	Exceedance frequency $F(P > a_i)$ $\frac{(N_i)}{n}$	Cumulative frequency $F(P < a_i)$ $1 - F(P > a_i)$	Return period	
	Lower limit (a_i)	Upper limit (b_i) incl.					T_{a_i} (days) $= \frac{1}{(6)}$	T_{a_i} (years) $= \frac{1}{30} (8)$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	(0-8	0) ⁺	285	0.50000	1.00000	0.00000	1	0.033
2	0	1	246	0.43158	0.50000	0.50000	2	0.667
3	1	2	25	0.04386	0.06842	0.93158	15	0.500
4	2	3	8	0.01404	0.02456	0.97544	41	1.367
5	3	4	3	0.00526	0.01053	0.98947	95	3.167
6	4	5	1	0.00175	0.00526	0.99474	190	6.333
7	5	6	0	0.00000	0.00351	0.99649	285	9.500
8	6	7	1	0.00175	0.00351	0.99649	285	9.500
9	7	8	1	0.00175	0.00175	0.99825	570	19.000

$k = 9$
 $n = \sum m_i = 570$

⁺ zero rainfalls
 $\delta = 0.0 \dots$

$$F(P > P_r) = \frac{r}{n+1} \tag{5}$$

- Calculate the frequency of non-exceedance

$$F(P \leq P_r) = 1 - F(P > P_r) = 1 - \frac{r}{n+1} \tag{6}$$

If the ranking order had been ascending instead of descending, similar relations as above would have been applicable with an interchange of $F(P > P_r)$ and $F(P \leq P_r)$. An advantage of using the denominator $n+1$ instead of n , as in Sect.2.1, is that the results will be identical no matter whether ascending or descending ranking orders are used.

Table 3 shows how the procedure has been applied for the monthly rainfalls of Table 1, and Table 4 for the monthly maximum 1-day rainfalls of Table 1.

Table 3. Frequency distributions based on depth ranking of monthly rainfalls derived from Table 1.

Rank number	Rainfall (descending)		Year	$F(P > P_r)$	$F(P \leq P_r)$	T_{P_r} (years)
r	P_r	P_r^2		$r/(n+1)$	$1-F(P > P_r)$	$\frac{1}{(5)}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	16.57	275	1962	0.05	0.95	20
2	12.29	151	1952	0.10	0.90	10
3	11.50	132	1958	0.15	0.85	6.7
4	10.93	119	1961	0.20	0.80	5.0
5	9.54	91	1951	0.25	0.75	4.0
6	9.52	91	1959	0.30	0.70	3.3
7	9.51	91	1964	0.35	0.65	2.9
8	9.45	89	1963	0.40	0.60	2.5
9	9.23	85	1953	0.45	0.55	2.2
10	9.20	85	1955	0.50	0.50	2.0
11	9.06	82	1957	0.55	0.45	1.82
12	8.76	77	1950	0.60	0.40	1.67
13	7.90	62	1966	0.65	0.35	1.54
14	6.66	44	1965	0.70	0.30	1.43
15	5.31	28	1948	0.75	0.25	1.33
16	4.48	20	1960	0.80	0.20	1.25
17	3.91	15	1956	0.85	0.15	1.18
18	2.51	6	1954	0.90	0.10	1.11
19	1.37	2	1949	0.95	0.05	1.05

$$n=19 \quad \sum_{r=1}^n P_r = 157.70 \quad \sum_{r=1}^n P_r^2 = 1545$$

Table 4. Frequency distributions, based on depth ranking, of maximum 1-day rainfalls per month, derived from Table 1.

Rank number	Rainfall (descending)		Year	$F(P > P_r)$	$F(P < P_r)$	T_{P_r} (years)
r	P_r	P_r^2		$r/(n+1)$	$1-F(P > P_r)$	$\frac{1}{(5)}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	7.93	63	1962	0.05	0.95	20
2	6.23	39	1961	0.10	0.90	10
3	4.37	19	1952	0.15	0.85	6.7
4	3.89	15	1951	0.20	0.80	5.0
5	3.64	13	1958	0.25	0.75	4.0
6	2.93	8.6	1963	0.30	0.70	3.3
7	2.65	7.0	1950	0.35	0.65	2.9
8	2.57	6.6	1966	0.40	0.60	2.5
9	2.55	6.5	1959	0.45	0.55	2.2
10	2.25	5.0	1955	0.50	0.50	2.0
11	2.21	4.9	1965	0.55	0.45	1.82
12	1.80	3.2	1957	0.60	0.40	1.67
13	1.78	3.2	1948	0.65	0.35	1.54
14	1.57	2.5	1964	0.70	0.30	1.43
15	1.51	2.3	1953	0.75	0.25	1.33
16	1.42	2.0	1954	0.80	0.20	1.25
17	1.17	1.4	1956	0.85	0.15	1.18
18	1.10	1.2	1960	0.90	0.10	1.11
19	0.40	0.2	1949	0.95	0.05	1.05

$$n=19 \quad \sum_{r=1}^n P_r = 51.97 \quad \sum_{r=1}^n P_r^2 = 203.6$$

18.2.3 RECURRENCE PREDICTIONS AND RETURN PERIODS

An observed frequency distribution of rainfalls can be regarded as a sample of the frequency distribution of the rainfalls that would occur in an infinitely long observation series (the population). If the sample is representative of the population, one may expect that future observation periods will reveal frequency distribution similar to the observed one. Hence an observed frequency distribution may be used for recurrence predictions.

It is a basic law of statistics that conclusions drawn for the population on grounds of a sample will be increasingly reliable as the size of the sample increases. Qualitatively it can be said that the smaller the frequency of occurrence of an event, the larger is the sample needed to obtain a prediction of the required accuracy. Referring to Table 2 it can be stated that the observed frequency of dry days (50%) will deviate only slightly from the frequency of dry days to be observed in a future period of at least equal length. The frequency of daily rainfalls of 3-4 inches (0.5%), however, may be doubled or reduced by half in the next period of record. A quantitative evaluation of the reliability of frequency predictions will be given in Sect.6.

Recurrence predictions are often done in terms of return periods (T), which is the number of new data one has to collect for a certain rainfall to be exceeded once on the average. The return period is calculated as

$$T = \frac{1}{F} \quad (\text{frequency of exceedance}) \quad (7)$$

In Table 2 the frequency of 1-day rainfalls in the interval of 1-2 inches equals 0.04386. Then the return period is $T = \frac{1}{F} = \frac{1}{0.04386} = \text{any 23 November days.}$

In hydrological practice one often works with frequencies of exceedance. The corresponding return period is symbolized as

$$T_x \approx \frac{1}{F(P > x)} \quad (8)$$

In design practice, T is often expressed in years

$$T(\text{years}) = \frac{T}{\text{no. of independent observations per year}} \quad (9)$$

For example, in Table 2, the frequency of exceedance of 1-day rainfalls of more than 4 inches in November is $F(P > 4) = 0.00526$. Thus the return period is

$$T_4 = \frac{1}{F(P > 4)} = \frac{1}{0.00526} = \text{any 190 November days.}$$

As daily rainfalls can generally be considered independent of each other, it follows, since there are 30 November days in one year, that

$$T(\text{years}) = \frac{T_4 (\text{November days})}{30} = \frac{190}{30} = 6.33 \text{ years}$$

In other words one has to wait on the average 6 years of 30 November days and 10 November days in the 7th year to find one November day on which the rainfall exceeds 4 inches.

In Fig.5, the rainfalls of Tables 2, 3, and 4 have been plotted against their respective return periods. Smooth curves have been drawn which fit the respective points as well as possible. These curves may be considered the most likely estimates of the population frequency (probability) distributions and of average future frequencies.

The smoothing procedure has the advantage that it makes interpolation possible and that it levels off random variations to a certain extent. It has the disadvantage that it may suggest an accuracy for prediction which does not exist. It is therefore useful to add confidence intervals for each of the curves in order to judge till which point the curve is reasonably reliable (Sect.6).

The frequency analysis illustrated in this section is usually adequate for problems related to agriculture. If there are approximately 20 years of information available, predictions of 10 year rainfalls will be possible, whilst predictions of 20 year rainfalls or more will be less reliable. It can be concluded that for return periods of 5 years or more there is no significant difference whether the analysis

Rainfall data

is carried out on the basis of all 1-day rainfalls or on maximum 1-day rainfalls only. This enables the analysis to be restricted to maximum rainfalls only, thus saving on labour but nevertheless obtaining virtually the same results for longer return periods.

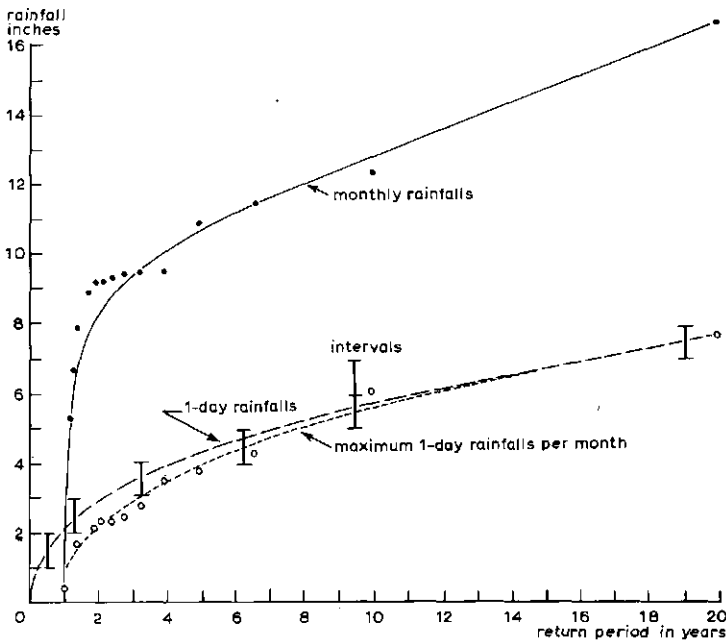


Fig.5. Depth return period relations derived from Tables 1,2,3, and 4

18.3 DURATION - FREQUENCY ANALYSIS OF RAINFALL

The time analysis is carried out, for the appropriate season, with respect to the duration of rainfall, which is chosen in accordance with the type of drainage problem (Sect.2).

The season is a period with fixed date limits at beginning and end, unlike duration, of which only the length is fixed. For a general determination of

critical season, average monthly rainfall values usually provide sufficient information (Fig.1). For water resources planning, seasonal frequency distributions may have to be established (Fig.2).

In analyzing the duration of rainfalls one encounters the difficulty of moving or sliding limits. When a 24-hour rainfall is studied, for example, this does not necessarily mean the rainfall from 8 o'clock one day till 8 o'clock next day, but the rainfall of any 24-hour period. If rainfall is measured at fixed intervals with pluviometers, it may happen that rainstorms are recorded in two parts. The frequencies of high 24-hour rainfalls will thus be underestimated. The drawback of interval measurements by pluviometers is avoided by making use of continuously recording rain gauges (pluviographs).

In the next parts of this section attention will be paid to the duration analysis for rainfalls which are measured at regular intervals with pluviometers. We distinguish:

- rainfalls equal to the interval of measurement (e.g.1 day)
- rainfalls of a duration composed of k intervals (k = 1,2, ... n).

Rainfall analysis for durations less than the interval of measurement, which is of importance for surface drainage of small steep watersheds with short critical durations, cannot be performed directly. In this case one has to resort to measurements with pluviographs or to generalized rainfall-duration relationships, obtained from pluviographs elsewhere (Sect.3.4).

18.3.1 FREQUENCY ANALYSIS FOR DURATIONS EQUAL TO THE INTERVAL OF MEASUREMENT

From the 1-day rainfalls presented in Table 1, the frequency distribution has been derived and presented in Table 2. With 30 November days in each year, the return period has been calculated as

$$T_x(\text{years}) = \frac{T_x(\text{days})}{30} = \frac{1}{30F(P>x)} \quad (10)$$

The frequencies of high 24-hour rainfalls are probably a little higher than for the 1-day (8 o'clock - 8 o'clock) rainfalls, because a rainstorm may have been recorded on two consecutive days. It has been found in the U.S.A. and The Netherlands that this effect can be compensated for by multiplying rainfall depths with a return period of approximately 10 years with a factor 1.1.

18.3.2 FREQUENCY ANALYSIS FOR DURATIONS COMPOSED OF TWO OR MORE INTERVALS OF MEASUREMENT

For durations composed of two or more intervals of measurement, there are various techniques of composition (Fig.6):

- successive totals
- moving totals
- maximum totals.

Examples of all three techniques will be given below, using 5-day durations, but the same reasoning can be applied for any other duration. Successive 5-day totals are formed by breaking the considered period or season into consecutive groups of 5-days and calculating the total rainfall for each group. These totals have the same drawback as have the fixed-hour observations in that they may split periods with high rainfall into two parts of lesser rainfall, thus underestimating frequencies of high rainfalls. A further disadvantage is that one obtains a long series of data of which only a part is of interest.

date	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20									
rainfall	0.02	0.60	0.20	0.80	2.20	2.50	0.85	0.60	0.00	0.25	0.40	1.05	1.30	0.20	0.10	0.00	0.00	0.00	0.22	0.35									
5 day successive totals	← 3.82 →				← 4.30 →				← 3.05 →				← 0.57 →																
5 day moving totals	← 3.82 →				← 6.40 →			← 6.65 →		← 7.05 →		← 2.10 →		← 2.30 →		← 3.00 →		← 3.20 →		← 3.05 →		← 1.60 →		← 0.30 →		← 0.32 →		← 0.57 →	
5 day maximum totals	← 7.05 →				← 4.30 →				← 3.05 →				← 2.65 →																

Fig.6. Illustration of various methods for the composition of 5-day totals

Moving 5-day totals are formed by adding to the rainfalls of each day in the considered period the rainfalls of the following 4 days. Thus one obtains an overlap, each daily rainfall being represented 5 times. Although in the month of November there are 26 moving 5-day totals to be composed, one can, for the calculation of the return period, account only for 6 independent 5-day totals, which is the same as for the successive totals. The advantage of the moving totals is that they take into consideration all possible 5-day totals and thus do not result in an underestimation of high 5-day rainfalls. A drawback is the very large number of data one has to compose and analyse, almost 5 times as much as for the successive totals, whereas a great part of the information may be of little interest.

It is for this reason that reduced data series are often used. In these, the data of less importance, e.g. low rainfalls when drainage projects are being considered, are omitted, and exceedance series or maximum series only are selected.

Maximum 5-day totals constitute the highest of the 5-day moving totals found for each year. A straightforward frequency distribution or return period distribution can then be made with the interval or ranking procedure. However, the second highest rainfall in a year may exceed the maximum rainfall recorded in some other years. As a consequence the rainfall depths with return periods of less than approximately 5 years will be underestimated in comparison with those obtained from complete or exceedance data series (Sect.7.4). Although, for high return periods, the difference between maximum series and complete series vanishes (see Fig.5), it is recommended, for agricultural purposes, not to work exclusively with maximum series.

Frequency analysis from pluviograph records

If pluviograph records are available in the form of unprocessed recorder charts, these charts will have to be processed either manually or mechanically to obtain lists of rainfall depths per unit of time (e.g.5,10,30 min.). When punch tape recorders are used on the pluviograph the processing can be entirely automated.

Once the lists of rainfall depths for various durations are available, the analysis can, in principle, be performed along the same lines as was discussed for daily records. There will, however, be such a mass of data that only maximum series will be practical.

18.3.3 AN EXAMPLE OF THE APPLICATION OF DEPTH-DURATION-FREQUENCY RELATIONS TO DETERMINE A DRAINAGE DESIGN DISCHARGE

Having analysed rainfalls for both frequency and duration, one arrives at the depth-duration-frequency relations as illustrated in Fig.3. These relations hold only for the point where the observations were made, but can be assumed representative of the area surrounding the recording station if the climate in the area is uniform.

An example of the application of depth-duration-frequency relations is given below. The data of Table 1 come from a tropical rice growing area. November, when rice has just been transplanted, is a critical month: an abrupt rise of more than 3 inches of the water standing in the fields is considered harmful. A surface drainage system is to be designed to prevent this happening too frequently. To find the design discharge of this surface drainage system, we make use of the

data of Table 1. We first determine the frequency distributions of 1,2,3, and 5-day rainfalls in the way discussed earlier in this section. From this analysis 1,2,3, and 5-day rainfalls with return periods of 5,10, and 20 years are selected and plotted (Fig.7).

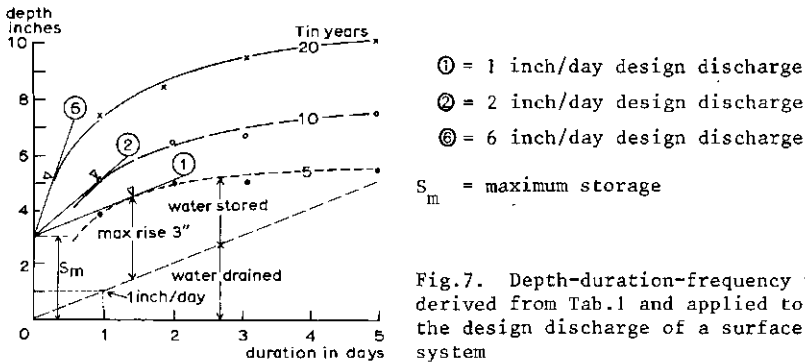


Fig.7. Depth-duration-frequency relation derived from Tab.1 and applied to determine the design discharge of a surface drainage system

To find the required design discharge in relation to the return period (accepted risk of inadequate drainage), we draw tangent lines from the 3 inch point on the rainfall axis to the various duration curves. The slope of the tangent line indicates the design discharge. If the tangent line is shifted parallel to pass through the zero point of the coördinate axes, we can see that, if a 5-year return period is accepted, the drainage capacity should be 1 inch/day. It can also be seen from Fig.7 that the maximum storage then amounts to 3 inches. If the design frequency is taken at 10 years, the discharge capacity would be 2 inches per day, whilst for 20 years it would be 6 inches per day. It can be seen that the critical durations, indicated by the tangent points, become shorter as the return period increases.

18.3.4 GENERALIZED DEPTH-DURATION-FREQUENCY RELATIONS

To carry out a rainfall frequency analysis as described in the foregoing sections, one must have rainfall records that cover a sufficient number of years. For durations longer than one day, this will usually be the case, but for durations less than one day the available rainfall records (pluviographs) may be limited or non-existent, and thus not allow a rainfall analysis. Recourse should then be made to generalized depth-duration-frequency relations such as:

- empirical formulas relating depth, duration, and frequency, as established for rainfall stations with sufficiently long records and considered representative of the location under study

- maps on which, for various durations and frequencies, the isopluvials are drawn, based on the long-record rainfall stations in the area.

The empirical formulas usually have two parameters, which may change from one area to another.

Examples of such formulas are

$$P = a \cdot t^b \quad (t > 2 \text{ hrs, frequency constant}) \quad (11)$$

$$P = \frac{c \cdot t}{t+d} \quad (t < 2 \text{ hrs, frequency constant}) \quad (12)$$

$$P = A \log T + B \quad (\text{duration constant}) \quad (13)$$

where a , b , c , d , A , and B are parameters to be determined empirically.

When rainfall maps are available, such as those in the Rainfall Frequency Atlas (HERSHFIELD, 1961), the rainfall depths for any particular location and a certain combination of duration and frequency can be read from them. The Rainfall Frequency Atlas gives twelve values, namely the 30-minutes, 1-hour, 6-hours, and 24-hours rainfall depths for return periods of 2 years, 10 years, and 100 years. Any other desired value can be obtained by linear interpolation, using special diagrams (Fig.8).

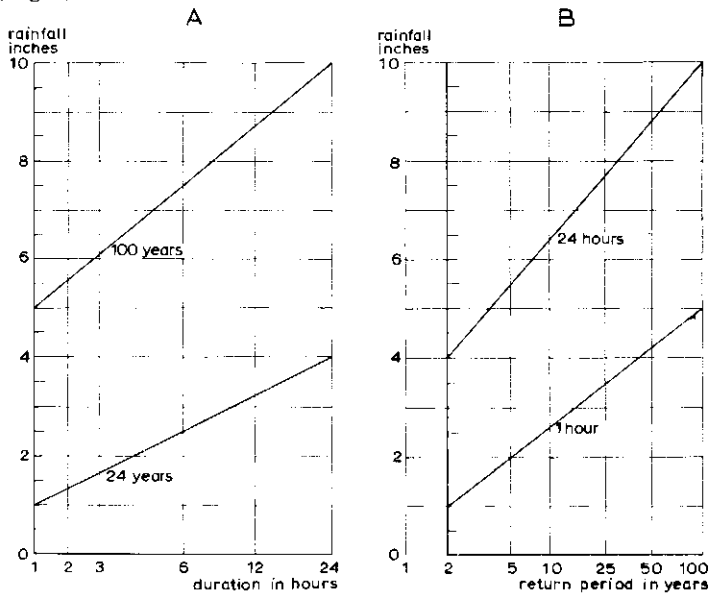


Fig.8. Generalized depth-duration frequency relations (Hershfield 1961).
 A: Rainfall depth-duration diagram. B: Rainfall depth versus return period.
Schematic example of use

Diagram A is used to interpolate between durations of 1-hr and 24-hrs, and Diagram B is used to interpolate between return periods of 1 year and 100 years. Diagram A is derived empirically, while Diagram B is based on the Gumbel probability distribution with a correction for partial-durations when the return period is less than 10 years.

For the linear interpolation on Diagrams A and B, four key-values only would suffice. REICH (1963), in a study on the short-duration rainfall intensities of South Africa, worked with the key-values: 1-hour and 24-hours duration rainfall depths for 2-year and 100-year return periods. If the key-values cannot be read directly from available rainfall maps, empirical relations are suggested to estimate the key-value from such information as: occurrence of thunder storms, mean of annual maximum daily rainfalls, and the ratio between 100-year maximum and 2-year maximum rainfalls as found from neighbouring recorder stations.

18.4 DEPTH-AREA ANALYSIS OF RAINFALL

The analysis of rainfall is understood here to be the analysis of area averages of point rainfalls or the analysis of the geographical distribution of point rainfalls. The first type of analysis will be given below, the second type has been discussed in Sect.3.4.

A rainfall measurement is a point observation and may not *a priori* be representative of the area. Usually area rainfalls have a smaller variability than point rainfalls (Fig.4B). For high return periods, this results in area rainfalls which are smaller than point rainfalls and vice versa. However, area rainfalls will differ less from point rainfalls if the duration is taken longer. Therefore mean rainfalls taken over long periods will be approximately equal for points and for areas, if the area is homogeneous with respect to rainfall. An area-frequency analysis can be made on the basis of the area averages of point rainfalls for the chosen duration. Usually one of the following three methods is employed:

- arithmetic mean of rainfall depths at all stations,
- weighted mean of rainfall depths at all stations, the weight being determined by polygons constructed according to the Thiessen method,
- weighted mean of average rainfall depths between isopluvial lines, the weight being the area enclosed by the isopluvials.

Examples of the above procedures are given in Fig.9.

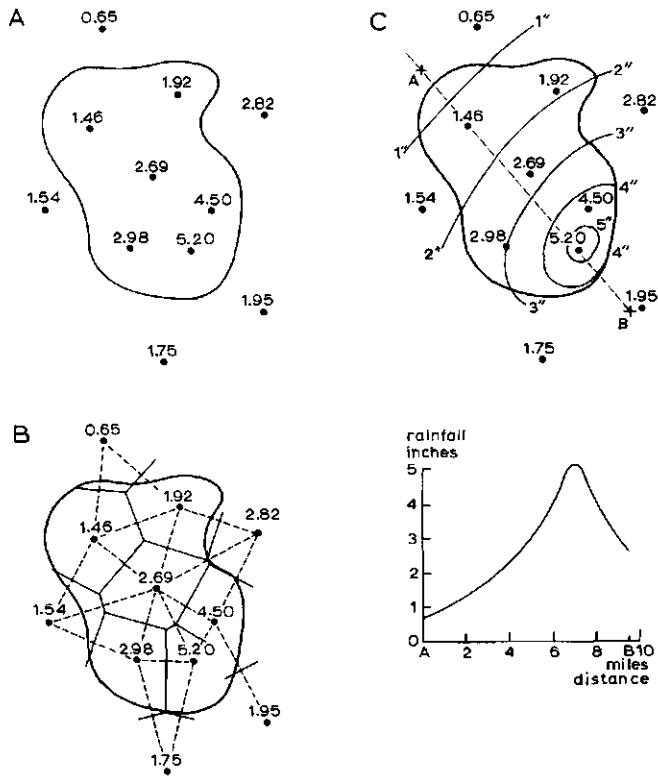


Fig.9. Methods of computing area averages of rainfall. A: Arithmetic mean method. B: Thiessen method. C: Isohyetal method

An advantage of the arithmetic mean is its simplicity. The method can only be used in a relatively flat area, where no irregular changes occur in isopluvial spacing and where the stations are evenly distributed, thus being equally representative. With this method the computation of the area average rainfall is as follows

$$\frac{1.46 + 1.92 + 2.69 + 4.50 + 2.98 + 5.20}{6} = 3.12 \text{ in.}$$

The Thiessen method assumes that the recorded rainfall in a station is representative of the area half-way to the adjacent stations. Each station is therefore connected with its adjacent stations by straight lines, the perpendicular bisectors of which form a pattern of polygons. The area which each station is taken

Rainfall data

to represent is the area of its polygon and this area is used as a weight factor for its rainfall. The sum of the products of station areas and rainfalls is divided by the total area covered by all stations to get the weighted average rainfall. With this method the computation of the area average rainfall is as follows:

Observed rainfall (in.)	Area ^{+) (sq.mi)}	Weighted rainfall col (1)×col (2)/626
0.65	7	0.01
1.46	120	0.28
1.92	109	0.35
2.69	120	0.51
1.54	20	0.05
2.98	92	0.45
5.20	82	0.68
4.50	76	0.54
	<u>626</u>	<u>2.87</u> in.

The Thiessen method can be used when the stations are not evenly distributed over the area. As, however, the method is rather rigid, excluding possible additional information on local meteorological conditions, its use is restricted to relatively flat areas.

When the rainfall is rather unevenly distributed over the area, for instance due to differences in orographic exposure, the isohyetal method may be applied to compute the area rainfall. This method consists of drawing lines of equal rainfall depth, isopluvials or isohyets, by interpolation between observed rainfall depths at the stations. Any additional information available may be used to adjust the interpolation. With this method the computation of the area average rainfall is as follows:

Isohyet	Rainfall between isohyets (in.)	Area ^{++) (sq.mi)}	Weighted rainfall col (2)×col (3)/626
5	5.1	13	0.10
4	4.5	77	0.55
3	3.5	116	0.65
2	2.5	196	0.78
1	1.5	193	0.46
<1	0.9	31	0.04
		<u>626</u>	<u>2.58</u> in.

^{+) Area of corresponding polygon within basin boundary}

^{++) Area enclosed by successive isohyets within basin boundary}

The area average rainfall is computed from the weighted mean of average rainfalls between two isohyets, the weight being the enclosed area between the isohyets. The reliability of the method depends on the accuracy with which the isopluvials can be drawn.

18.5 SURVEY AND MEASUREMENT

18.5.1 MEASUREMENT OF POINT RAINFALL

Rainfall is measured with rain-gauges, which are provided with a receiver having a horizontal opening of known area (Fig.10). In principle any open receptacle may serve as a rain-gauge. Commonly used are receivers with an opening of 200 to 400 cm². The rainfall collected in the gauge is funnelled into a small-diameter measuring tube, where its depth is amplified so that tenths of a millimeter can be measured accurately. A distinction can be made between two types of rain-gauges: self-recording and non-recording.

Non-recording gauges or pluviometers are measured by periodical readings of the accumulated rainfall. This is generally done every 24 hours. Large capacity storage gauges, to be used at remote sites, are measured only a few times a year. Pluviometers have the advantage that they can be made simple and sturdy; consequently they are reliable and not very costly.

They have, however, certain disadvantages:

- they require personnel for reading,
- the fixed hour readings may not correspond with the occurrence of rain-storms, which will then be divided over two consecutive intervals (see Section 3.1),
- the distribution of rainfall within the interval of observation remains unknown.

Recording gauges or pluviographs give continuous recordings of the rain being caught in the gauges. They enable the rainfall depth over any period to be read and are a prerequisite if short duration rainfalls have to be determined. If recordings are made on punch cards or magnetic tapes, these records can be fed into electronic computing equipment without further processing.

Their disadvantages are:

- they are more complicated and more expensive than pluviometers,
- they produce an almost unlimited amount of information which may be more than required, making data selection procedures necessary.

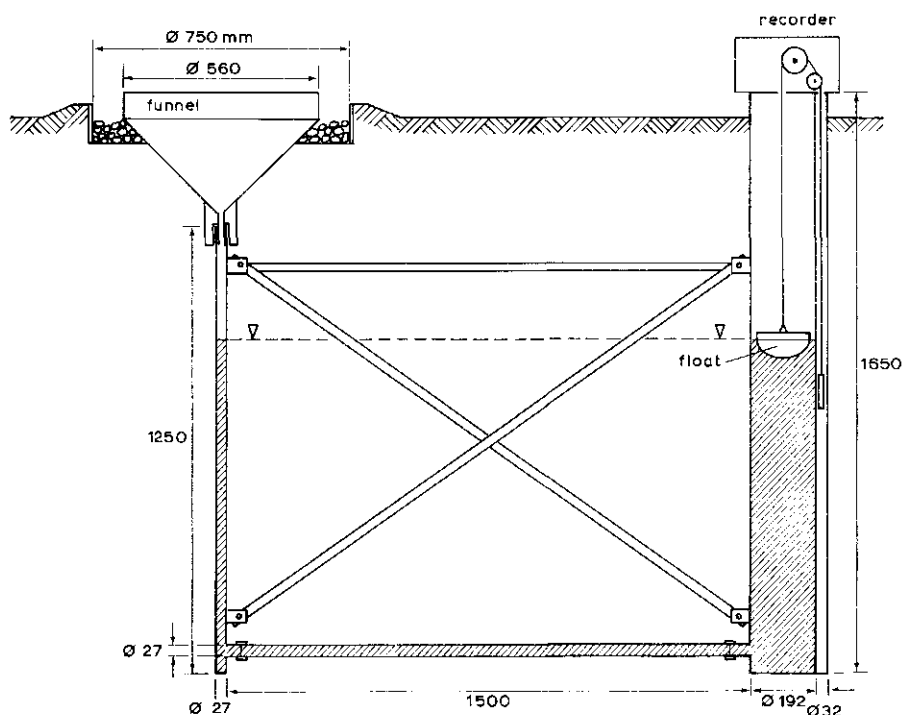


Fig.10. Cross-section of the Colenbrander rain-gauge

Although the principle of measuring precipitation is relatively simple, it is sometimes difficult to obtain accurate measurements. There are various influences which make rain-gauge readings uncertain, such as

- the wind shelter effect with respect to surrounding objects,
- the wind deformation effect with respect to the rain-gauge itself,
- the evaporation losses.

The wind shelter effect occurs when the rain-gauge is too near a house, a tree, etc. It may give rise to an excess or deficit in catch. The World Meteorological Organization (Guide to Hydrometeorological Practices, 1965) gives as a rule of thumb that the gauge should not be closer to surrounding objects than 4 times the height of these objects.

The wind deformation effect is due to the fact that wind reduces the amount of rain caught in the receiver by blowing the rain drops upward over the edge of the receiver. It has been observed that wind can reduce the catch by more than 10% (McKay, 1954). The wind deformation effect is determined by the size of the

opening and the height of the receiver above the ground surface. Since the wind deformation effect occurs at the edges of the receiver, it can be reduced by making a larger opening by which the ratio circumference to surface becomes smaller. The height of the mouth of receiver above the ground should be as low as possible because the wind velocity increases with height, but should be high enough to prevent splashing in. In areas which have little snow and where the surroundings are such that, even in heavy rain, there is no risk of the ground being covered by puddles, a height of 30 cm is used. Otherwise a standard height of 1 metre is advised by the W.M.O. Standard heights in the USSR, the USA, The Netherlands, and the UK, are 2, 0.75, 0.40, and 0.30 respectively.

The deformation effect can be reduced by choosing the site so that the wind speed is as low as possible and also by modifying the surroundings of the gauge so that the airflow across the mouth is made accurately horizontal. This can be achieved by building a circular turf wall about 3 metres across (English mounting). The gauge is exposed in the centre of the circle. The inner surface of the wall should be vertical and the outer surface sloping at an angle of about 15% to the horizontal. An alternative way is to use shields around the receiver, with the top of the shield on the same level as the mouth of the receiver. Well known are the flexible Alter shield and the rigid Nipher shield.

Evaporation losses are likely to occur in hot and dry climates. They can be reduced by placing oil in the receiver and by preventing the internal temperature of the receiver from becoming too high. The equipment and practices for rainfall measurements are standardized in most countries. It is usually advisable to follow these standards, even when they would give rise to systematic errors, in order to facilitate a comparison between newly installed and already existing stations. When more accurate precipitation data are desirable, as in waterbalance studies, drainage experiments etc., a second gauge with a better performance could be installed next to the standard type, for comparison.

18.5.2 NETWORK DENSITY OF RAIN GAUGES

The required network density of rain gauges depends on the area variability of rainfall and the required accuracy. A relatively sparse network of stations would suffice for studies of long-duration rainfalls (e.g.annual). A very dense network is required to determine the rainfall pattern of thunderstorms. As rainfall characteristics in mountainous areas are more variable than in flat areas, the network density has to be higher in the first case than in the second.

Rainfall data

The accuracy with which the average area rainfall can be determined depends on the number (n) of stations. The standard error of the mean area rainfall decreases proportionally with $1/\sqrt{n}$, if the station records are independent. As apparent from Fig.11, the larger the area under consideration, the lower the network density required to determine the average area rainfall.

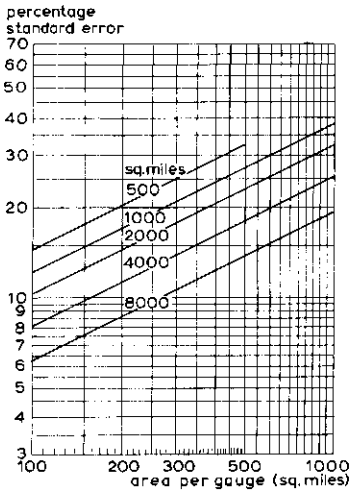


Fig.11. Standard error of average area precipitation as a function of network density and drainage area for the Muskingum Basin (U.S. Weather Bureau)

18.6 RELIABILITY OF RECURRENCE PREDICTIONS

The reliability of recurrence predictions depends, among factors such as variability, on the number of observations. The reliability can be expressed quantitatively by confidence intervals. An $\alpha\%$ confidence interval of a statistical variate y is defined as the interval $a-b$ for which there is $\alpha\%$ probability that $a < y < b$.

For calculated frequencies, the determination of confidence intervals can be made with the binomial probability distribution. This distribution deals with two possibilities only: an event occurs or does not occur, a rainfall depth is exceeded or is not exceeded. If the probability of exceedance of a rainfall P_r is p and the probability of non-exceedance is $q = 1 - p$, then the probability that the exceedance occurs m times when n independent observations are made is

$$\text{Prob}(m) = \frac{n!}{m! (n-m)!} p^m q^{(n-m)} \tag{14}$$

which is the binomial probability distribution of m , where the symbol $!$ stands for factorial according to

$$n! = n.(n-1).(n-2) \dots 3.2.1$$

$$m! = m.(m-1).(m-2) \dots 3.2.1$$

Since the frequency of exceedance is derived from a limited amount of data, it is only an estimate of the probability of exceedance p . By applying the binomial confidence interval to the observed frequency of exceedance, one obtains the interval that covers p at the chosen confidence level. A series of confidence belts of p is obtained as depicted in Fig.12 for various values of sample size n .

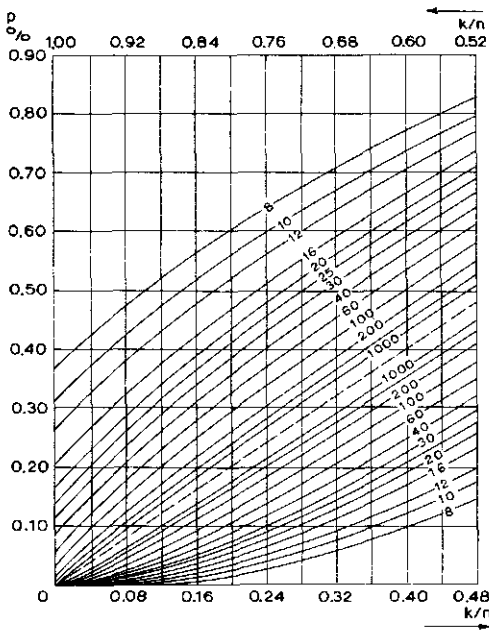


Fig.12. 95% confidence limits derived from the binomial distribution (Pearson, Hartley, 1956)

This figure allows for statements such as:

"It is for 95% likely that the probability of exceedance of a daily rainfall of 1 inch for which in a sample of size $n = 570$ the observed frequency of exceedance was $F(P > 1 \text{ inch}) = 0.07$ (see Table 2) is covered by the interval 0.05 (lower limit) and 0.10 (upper limit)." The return period ranges in this case from 10 to 20. In literature methods are given for the determination of confidence intervals for the entire probability distribution.

Recording stations often have lengths of records which are too short for sufficiently reliable predictions.

The records may be extended in two ways:

- by relating the record to the records of neighbouring stations with longer record (e.g. by a linear regression or by the ratio method). This method can only be successful if there is a good correlation. Further reference on this subject is made to SNEDECOR (1957);

- by adding the records of one or more stations (station year method). This can only be done if the data are independent, which can be tested, for instance, by a correlation analysis. Also the stations should be in areas which, from the point of view of rainfall, are homogeneous. This can be tested by analyzing their rainfall distributions. Further reference on this subject is made to VEN TE CHOW (1964).

18.7 EXAMPLES OF DISTRIBUTION FITTING AND CONFIDENCE BELTS

18.7.1 INTRODUCTION TO DISTRIBUTION FITTING

When a frequency distribution has been obtained, it is considered representative for frequencies in the future. However, a frequency distribution of rainfalls is obtained from a data series of limited length, which does not contain all possible rainfall depths (the population). In other words the data series is only a sample of the population. Thus the frequency distribution of the data series is only an estimate for the frequency distribution of the population (the probability distribution). The degree to which an estimate can deviate from the population value is dependent on the sample size: the larger the sample, the smaller the deviation tends to be. This is a fundamental law of statistics.

The probability distribution is, for rainfalls, assumed to be a continuous function, representing a smooth curve on a graph. A frequency distribution is, as a consequence of defectiveness of the data, discontinuous and not smooth. A first step towards finding a smooth probability distribution from a frequency distribution is by plotting the data on a graph against their frequencies (plotting positions) and drawing through the points the best fitting smooth curve (Fig.5).

There are, however, probability distributions available, described by mathematical expressions. Statistical experience has proved the general usefulness of these probability distributions. From this experience stems the practice in hydrology of determining the probability distribution and its parameters to fit the observed frequency distribution. In the following sections, a normal and a Gumbel probability distribution will be fitted to the data of Table 2.

For recurrence prediction a fitted probability distribution has certain advantages above the observed frequency distribution:

- it levels off random errors to a certain degree,
- it makes straight line plotting possible by a mathematical transformation,
- it provides a more or less reliable tool to extrapolate frequencies, or in other words to predict frequencies of rare events with return periods longer than the observation period,
- it provides a check on outlying observations.

18.7.2 A LOGARITHMIC TRANSFORMATION

As an example of the procedure of empirically fitting, the data of Table 2 are plotted in Fig.13 as the logarithm of rainfall depth versus the logarithm of the return period in years. A straight-line relationship is assumed. The line in Fig.13 is a graphical eye estimate of best fit.

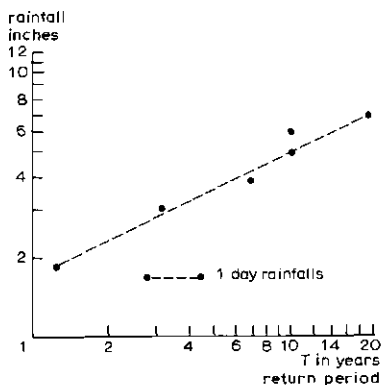


Fig.13. Depth-return period relation for 1-day rainfall totals derived from Table 2 and plotted on double logarithmic paper

18.7.3 THE NORMAL PROBABILITY DISTRIBUTION

Theory

The normal probability distribution, also known as the Gauss or the Moivre distribution, cannot be written directly in terms of probabilities. The analytical expression is given in the form of a probability density function

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(x-\mu)^2}{2\sigma^2}} \quad (15)$$

where

- $f(x)$ = the normal probability density function of x
- x = the normal variate ($-\infty < x < \infty$)
- μ = the mathematical expectation of x
- σ^2 = the variance of x

A probability of occurrence in a certain interval can be found by

$$\text{Prob}(a < x < b) = \int_a^b f(x) dx \quad (16)$$

Since

$$\int_{-\infty}^{+\infty} f(x) dx = 1 \quad (17)$$

it follows that

$$\text{Prob}(x > a) = 1 - \text{Prob}(x \leq a) \quad (18)$$

The mathematical expectation of x is given by:

$$\mu = \int_{-\infty}^{\infty} x f(x) dx \quad (19)$$

The variance, a measure of the variation of x about μ , is given by

$$\sigma^2 = \int_{-\infty}^{\infty} (x-\mu)^2 f(x) dx \quad (20)$$

In Fig.14A an example is given of a normal probability density function. It can be seen from the formula that the density function is symmetric about μ . The mode u (where the function is maximum) coincides with the mean μ . The probability of both exceedance and non-exceedance of μ equal $\frac{1}{2}$. The median ε therefore also coincides with the mean.

If $\mu = 0$ and $\sigma = 1$ the distribution is called standardized normal, of which the variate y is an example

$$f(y) = \frac{1}{\sqrt{2\pi}} e^{-\frac{y^2}{2}} \quad (21)$$

Probabilities of it are found in tabulated form in statistical handbooks, e.g. FRAZER (1958). The tables have been prepared by numerical integration. The transformation $x = \mu + \sigma y$ or $y = \frac{x-\mu}{\sigma}$ makes it possible to apply the tabulated standardized distribution to any other normal distribution. Fig.14B shows the graph of the standardized normal probability function, which is tabulated in Table 5.

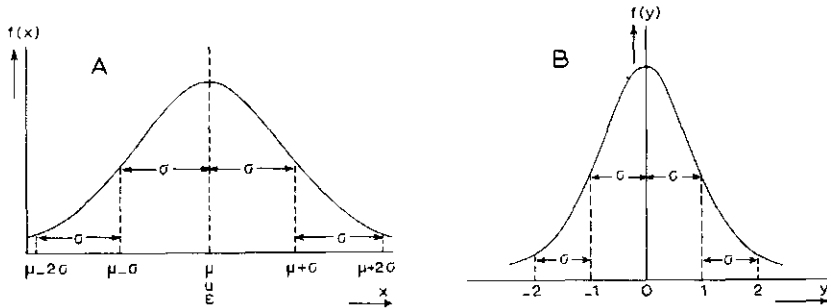


Fig.14. Examples of some normal distributions with some common properties. A: Normal probability density function. B: Standardized normal probability density function

Other transformations, of a more complicated nature, are sometimes performed on the normal distribution. An example is the lognormal distribution, which is a transformed normal distribution $f(z)$ where $z = \log x$. The further transformation of the original data by the introduction of more parameters (modified lognormal distribution) generally involves a cumbersome process of trial and error to find the right transformation and has, therefore, less application.

It can be proved that the arithmetic mean (\bar{x}_n) of a sample of size n , whatever the population distribution of x is, approaches a normal distribution with increasing n (central limit theory). An annual rainfall, being the sum of 365 daily rainfalls x_1 , equals $365 \bar{x}$. Therefore, since n is large, annual rainfalls are generally normally distributed. In most cases the same holds for monthly rainfalls. Rainfalls of shorter duration have generally a skew distribution.

If a sample series is available for a variate which is assumed to have a normal distribution, the parameters μ and σ can be estimated from the sample, according to

$$\text{Est } (\mu) = \bar{x} = \frac{\sum x}{n} \tag{22}$$

(the arithmetic mean)

and

$$\text{Est } (\sigma) = s = \sqrt{\frac{\sum (x-\bar{x})^2}{n-1}} = \sqrt{\frac{\sum x^2 - n(\bar{x})^2}{n-1}} \tag{23}$$

(the standard error)

It can be proved that the standard deviation $\sigma_{\bar{x}_n}$ of the arithmetic mean \bar{x}_n of a sample is smaller than the standard deviation σ_x of the distribution of the individual values.

$$\sigma_{\bar{x}_n} = \frac{\sigma_x}{\sqrt{n}} \tag{24}$$

Table 5. The normal distribution $y \rightarrow f(y)$

y	0	1	2	3	4	5	6	7	8	9
0,0	5000	4960	4920	4880	4840	4801	4761	4721	4681	4641
0,1	4602	4562	4522	4483	4443	4404	4364	4325	4286	4247
0,2	4207	4168	4129	4090	4052	4013	3974	3936	3897	3859
0,3	3821	3783	3745	3707	3669	3632	3594	3557	3520	3483
0,4	3446	3409	3372	3336	3300	3264	3228	3192	3156	3121
0,5	3085	3050	3015	2981	2946	2912	2877	2843	2810	2776
0,6	2743	2709	2676	2643	2611	2578	2546	2514	2483	2451
0,7	2420	2389	2358	2327	2296	2266	2236	2206	2177	2148
0,8	2119	2090	2061	2033	2005	1977	1949	1922	1894	1867
0,9	1841	1814	1788	1762	1736	1711	1685	1660	1635	1611
1,0	1587	1562	1539	1515	1492	1469	1446	1423	1401	1379
1,1	1357	1335	1314	1292	1271	1251	1230	1210	1190	1170
1,2	1151	1131	1112	1093	1075	1056	1038	1020	1003	985
1,3	0968	0951	0934	0918	0901	0885	0869	0853	0838	0823
1,4	0808	0793	0778	0764	0749	0735	0721	0708	0694	0681
1,5	0668	0655	0643	0630	0618	0606	0594	0582	0571	0559
1,6	0548	0537	0526	0516	0505	0495	0485	0475	0465	0455
1,7	0446	0436	0427	0418	0409	0401	0392	0384	0375	0367
1,8	0359	0351	0344	0336	0329	0322	0314	0307	0301	0294
1,9	0287	0281	0274	0268	0262	0256	0250	0244	0239	0233
2,0	0228	0222	0217	0212	0207	0202	0197	0192	0188	0183
2,1	0179	0174	0170	0166	0162	0158	0154	0150	0146	0143
2,2	0139	0136	0132	0129	0125	0122	0119	0116	0113	0110
2,3	0107	0104	0102	0099	0096	0094	0091	0089	0087	0084
2,4	0082	0080	0078	0075	0073	0071	0069	0068	0066	0064
2,5	0062	0060	0059	0057	0055	0054	0052	0051	0049	0048
2,6	0047	0045	0044	0043	0041	0040	0039	0038	0037	0036
2,7	0035	0034	0033	0032	0031	0030	0029	0028	0027	0026
2,8	0026	0025	0024	0023	0023	0022	0021	0021	0020	0019
2,9	0019	0018	0018	0017	0016	0016	0015	0015	0014	0014
3,0	0013	0013	0013	0012	0012	0011	0011	0011	0010	0010
3,1	0010	0009	0009	0009	0008	0008	0008	0008	0007	0007
3,2	0007	0007	0006	0006	0006	0006	0006	0005	0005	0005
3,3	0005	0005	0005	0004	0004	0004	0004	0004	0004	0003
3,4	0003	0003	0003	0003	0003	0003	0003	0003	0003	0002

Procedure and example

As an example of the use of the normal distribution, the monthly totals of Table 3 have been plotted on normal probability paper of which the probability axis is constructed in such a way that the cumulative normal distribution

$$F(P < x) = \int_{-\infty}^x f(x) dx$$

is presented as a straight line (Fig.15). The plotting positions obtained from Table 3 and shown in Fig.15 do not produce a straight line, which is due to the small number of data available.

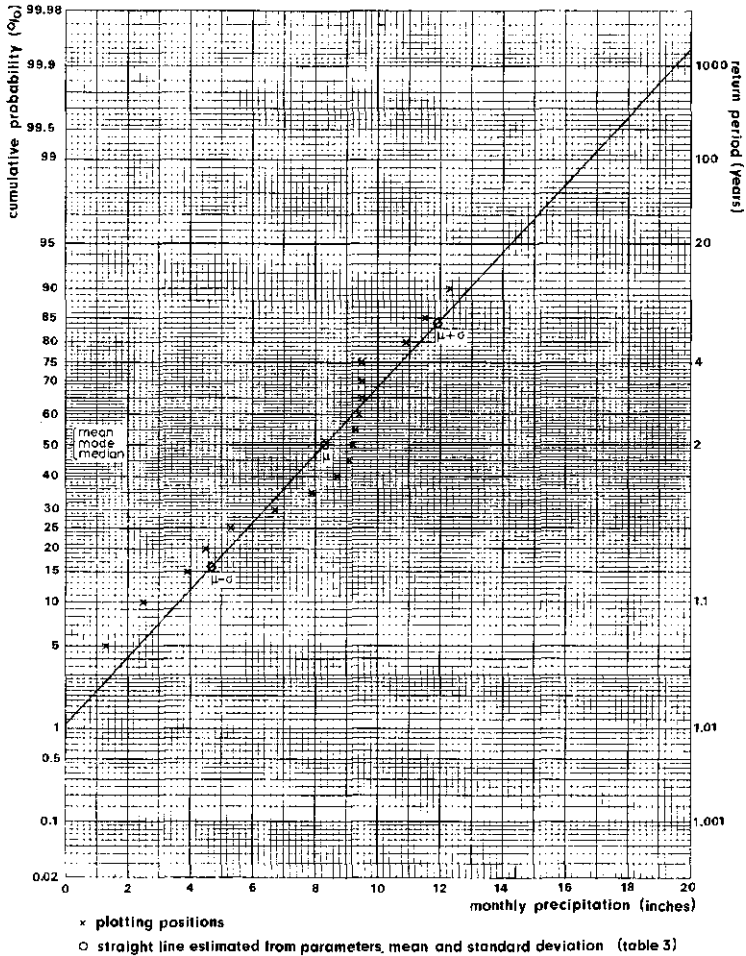


Fig.15. Monthly rainfalls as derived from Table 3 plotted on normal probability paper

In order to obtain a straight line, the parameters are estimated as $\mu = \bar{P}$ and $\sigma = s_p$ from Table 3, according to

$$\text{Est } (\mu) = \bar{P} = \frac{\sum P}{n} = \frac{157.70}{19} = 8.3$$

$$\text{Est } (\sigma) = s_p = \sqrt{\frac{\sum P^2 - n(\bar{P})^2}{n-1}} = \sqrt{\frac{1545 - 19(8.3)^2}{18}} = 3.6$$

The values $P = \mu - \sigma = 8.3 - 3.6 = 4.7$ and $P = \mu + \sigma = 8.3 + 3.6 = 11.9$ are then plotted against the 16% and 84% non-exceedance probability (see also Fig.14), whereas the value $P = \mu = 8.3$ is plotted against the 50% probability. A straight line can be drawn through these points.

It is concluded from Fig.15 that the observed monthly total of 16.57 inches is estimated to have a return period of approximately 100 years instead of the 20 years given as the observed frequency in Table 3. The first estimate (100 years) depends on the entire sample.

18.7.4 THE GUMBEL PROBABILITY DISTRIBUTION

Theory

The Gumbel distribution (GUMBEL, 1954), also called the distribution of extreme values can be written as a cumulative probability distribution, viz.

$$\text{Prob } (P_N < P) = \exp \{-\exp(-y)\} \quad (-\infty < P < \infty) \quad (25)$$

where

P_N = the maximum rainfall of a sample of size N ,

$y = \alpha(P-u)$ is the reduced Gumbel variate

$u = \mu - c/\alpha$ = the mode of the Gumbel distribution

μ = the population mean

c = Euler constant = 0.57722

$\alpha = \frac{\pi}{\sigma\sqrt{6}}$ = the standard deviation of the Gumbel distribution

σ = standard deviation of the population

The Gumbel distribution is skew to the right with $u < \mu$ and the median ε in between.

For large samples ($n > 100$, say) the parameters u and α can be estimated by means of the relationships

$$\text{Est } (\alpha) = \frac{\pi}{s_p \sqrt{6}} \quad (26)$$

and

$$\text{Est } (u) = \bar{P} - \frac{c}{\text{Est}(\alpha)} \quad (27)$$

Introducing $P=u$ into the equation of the Gumbel distribution yields, with $y = \alpha(P-u) = 0$

$$\text{Prob } (P_N < u) = e^{-1} = 0.37 \quad (28)$$

Therefore the probability of non-exceedance of the mode u equals 0.37.

It can be proved that the cumulative probability distribution of the maximum value of a sample of size N approaches asymptotically the Gumbel distribution with increasing N , if the samples are drawn from a distribution of the exponential type. In hydrological practice it can be assumed that the asymptotic approach is realized for $N > 10$; therefore frequent use is made of the Gumbel distribution in hydrology, mostly for annual or monthly maxima of floods or rainfalls of short duration (less than $\frac{1}{10}$ of year or month resp.).

For the determination of the Gumbel distribution one needs to have several samples from which to select the maxima, each sample being homogeneous or from the same population. In this way, annual, monthly, or seasonal maxima series for various durations can be composed, the duration being such that at least $N = 10$ independent data appear from which to choose the maximum.

Observed frequencies of non-exceedance for each of the n maxima selected from the $n \times N$ data can be obtained by using the ranking method (Sect.4.3) yielding

$$F(P_N < P_r) = 1 - F(P_N > P_r) = 1 - \frac{r}{n+1} \quad (\text{plotting position}) \quad (29)$$

where r is the rank number of the selected maxima, when arranged in decreasing order.

The Gumbel distribution can be written, taking logarithms twice

$$y = \alpha(P-u) = -\log \{-\log \text{Prob}(P_N < P)\} \quad (30)$$

Gumbel probability paper is constructed in such a way that probabilities of non-exceedance are plotted on a log log scale to yield a linear relationship. A straight line of best fit can thus be drawn by eye or can be calculated.

One of the methods for calculation is a linear regression. This regression is based on the linear relationship between y and P :

$$y = \alpha_1(P-u_1) \quad \text{regression of } y \text{ on } P \quad (31)$$

$$P = \frac{1}{\alpha_2} y + u_2 \quad \text{regression of } P \text{ on } y \quad (32)$$

The coefficients α_1 and $1/\alpha_2$ are estimated in such a way that the sum of the squares of the deviations in respectively y and P direction from the straight line is reduced to a minimum. Setting the geometric mean according to $\alpha = \sqrt{\alpha_1\alpha_2}$, it can be proved that

$$\text{Est}(\alpha) = \frac{s_n}{s_p} \quad \text{and} \quad \text{Est}(u) = \bar{P}_n - \frac{\bar{y}_n}{\text{Est}(\alpha)} \quad (33)$$

which determines the straight line for any value of n .

- s_p = is the standard deviation of the n maxima P_r
- s_n = is the standard deviation of n reduced variates y_r and depends only on n (see Table 6)
- \bar{P}_n = is the arithmetic mean of the n maxima P_r
- \bar{y}_n = is the arithmetic mean of n reduced variates y_r and depends only on n (see Table 6)

Procedure and example

As an example of the use of Gumbel distribution to a small sample, the maximum monthly 1-day rainfalls of Table 1 have been analyzed and are presented in Table 4. The cumulative frequencies have been plotted on Gumbel probability paper in Fig.16 together with confidence belts as derived from a specific method by Gumbel. For this example use is made of the regression analysis and Table 5, from which it follows that

$$\text{Est } (\alpha) = \frac{s_n}{s_p} = \frac{1.06}{1.84} = 0.58$$

and

$$\text{Est } (u) = \bar{P}_n - \frac{\bar{y}_n}{\text{Est}(\alpha)} = 2.74 - \frac{0.52}{0.58} = 1.84$$

Substitution of the above estimates into the equation $y = \alpha(P-u)$ gives

$$y = 0.58 (P - 1.84)$$

This is an expression for a straight line on Gumbel probability paper. Determination of two arbitrary points gives

$$y = 0 \rightarrow P = u = 1.84$$

$$y = 3 \rightarrow P = 7.00$$

which points have been plotted in Fig.16 and determine the estimate of the Gumbel distribution by regression.

Table 6. Expected means (\bar{y}) and standard deviations (s) of reduced extremes as a function of the sample size (n)

n	s_n	\bar{y}_n
10	0.950	0.495
15	1.021	0.513
20	1.063	0.524
25	1.092	0.531
30	1.112	0.536
40	1.141	0.544
50	1.161	0.548
70	1.185	0.555
100	1.206	0.560

$$\infty \quad 1.282 \left(= \frac{\pi}{\sqrt{6}} \right) \quad 0.577 \left(= c, \text{ Euler} \right)$$

A 1-day rainfall of 8 inches or more as a maximum in the month of November is thus estimated to have a return period of 38 years instead of 20 according to Table 4. The confidence belts drawn in Fig.16 by Gumbel's method, however, indicate that the population value of this return period can be expected to be covered by the interval given with 68% confidence probability. This interval can be made smaller only by making more observations.

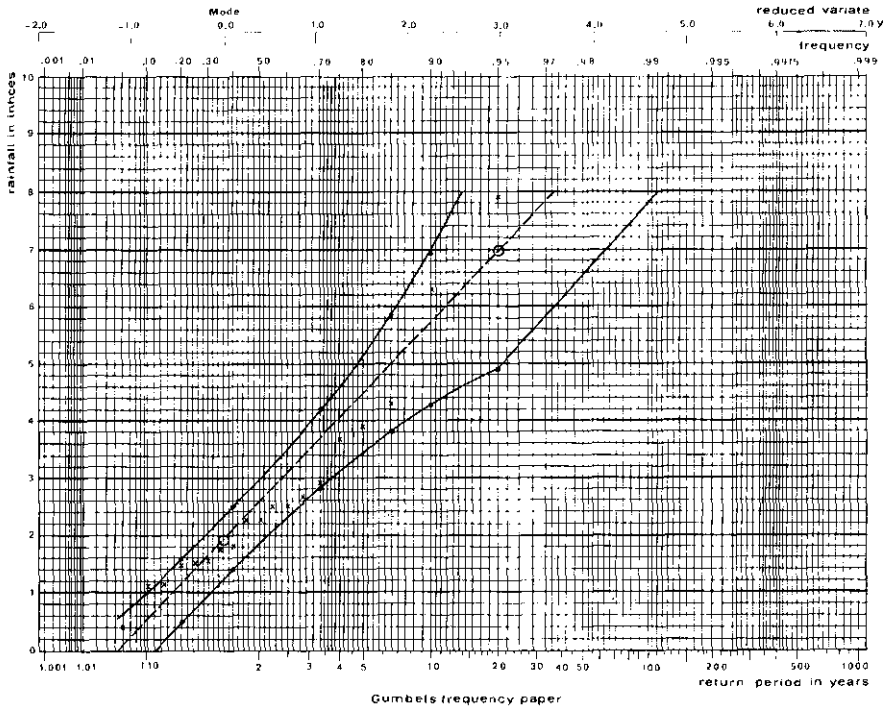
The Gumbel method allows for such statements as: on an average of once in T years there is one year with at least one rainfall equalling or exceeding a given depth. A limitation of the annual-maxima series is that each period is represented by only

Rainfall data

one event. The second highest event in a particular period would not be counted. To avoid these restrictions partial duration series (truncated series, see Sect.5) can be employed for which data that are greater than a certain base value are selected. When this base value is chosen in such a way that the number of values in the partial duration series is equal to the number of years of record, the series is called annual exceedance series. Table 7 serves to give an approximation of the corresponding return periods of annual exceedance series and annual maxima series.

Table 7. Corresponding return periods of annual exceedance series and annual maxima series

exceedance series	maxima series
0.50	1.16
1.00	1.58
1.45	2.00
2.00	2.54
5.00	5.52
10.00	10.50



- x frequency polygon for data from table 4
- O determined by Gumbels method, going line of best fit
- 68 % confidence interval by Gumbels method

Fig.16. Monthly maximum 1-day rainfalls as derived from Table 4 plotted on Gumbel probability paper

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SURVEYS AND INVESTIGATIONS

19. DETERMINING EVAPOTRANSPIRATION

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PURPOSE AND SCOPE

A description of the process of evapotranspiration and some methods for its calculation.

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19.1 INTRODUCTION

The various processes through which water can be added or removed from the root-zone of the soil are intimately related to one another because of the physical requirement of conservation of matter. All additions of water by infiltration from rainfall or irrigation or by capillary rise from the groundwater table, and all losses of water by evaporation, transpiration, or deep percolation, need to be accounted for in a water balance equation. Water balance studies have been used in water resources investigations, streamflow studies including river forecasting, irrigation operations, and in the derivation of maps of mean annual rainfall, runoff, and evapotranspiration. (Evapotranspiration is the combined effect of evaporation of water from moist soil and transpiration of water by a growing crop.) In applying the water balance it is usually necessary to make assumptions with respect to at least one of the variables. In many estimates of evapotranspiration in irrigated areas, for example, the drainage term is assumed to be negligible. It is important that these assumptions not be overlooked particularly when methods are being transferred from one field of investigation to another. Another important consideration is the accuracy with which each term of the water balance can be evaluated. In this chapter we are concerned with the evapotranspiration term in the water balance, the objective being to assess the various methods of estimating or measuring evapotranspiration, in order that some guidance can be given to those who design drainage systems. The soil, the plants, and the atmosphere are all components of a physically unified and dynamic system in which various flow processes occur interdependently like links in a chain (COWAN, 1965). In this system flow takes place from higher to lower potential energy, with the concept of water potential being equally valid and applicable in the plant, soil, and atmosphere (Chap.5, Vol.I). Spontaneous flow is in the direction of the negative potential gradient, i.e. from places with a relatively high potential (a smaller negative quantity) to one where the water potential is relatively low.

The quantity of water transpired daily is large relative to the change in water content of the plant, so we can treat flow through the plant for short periods as a steady state process. The potential differences in different parts of the system are then proportional to the resistance to flow. The resistance is generally greater in the soil than in the plant, and greatest in the transition from the leaves to the atmosphere, where water changes its state from liquid to vapour, water then moving out of the leaves through the stomata by the process of diffusion. The total potential difference between the soil and the atmosphere can amount to hundreds of bars, and in an arid climate can even exceed 1,000 bar.

The potential drop in the soil and plant is generally less than 10 bar so that the major portion of the overall potential difference occurs between the leaves and the atmosphere. Water transport from the leaves to the bulk air occurs as diffusion of water vapour, which is proportional to the water vapour pressure gradient. The relative humidity (i.e. the ratio of actual to saturation vapour pressure), is, in turn, exponentially related to the water potential. A distinction may be made between vapour transfer from a free water surface, from a bare soil surface, and from transpiring vegetation, but all of these processes are essentially similar, depending as they do on a supply of water and an energy source. The nature of the surface, however, does exert some influence on the rate of vapour loss because of various resistances and "shape" factors associated with plants, soils, and water bodies. We will return to these differences later.

The following terms are relevant to the discussion in this chapter and need to be defined:

- potential evapotranspiration, E_p , is the maximum amount of vapour which could be transferred from an area to the atmosphere under the existing meteorological conditions

- actual evapotranspiration, E_a , is the actual amount of vapour transferred to the atmosphere, which depends not only on existing meteorological conditions, but also on the availability of water to meet the atmospheric demand and, in the case of vegetation, its ability to extract moisture from the soil.

The water loss from a large area in which soil moisture is not a limiting factor is at the potential rate, and any variations across the area would result from differences in meteorological conditions, including advective effects. However, once drying out commences, the picture increases in complexity: the decrease of the actual evapotranspiration rate below the potential then depends on the available moisture in the soil, the rooting depth of the crop, and other plant characteristics. In extreme cases, the evapotranspiration demand of the atmosphere cannot be met by the crop even though soil moisture is not limiting. To calculate the expected amount of deep percolation in drainage design, one is usually interested in the potential evapotranspiration rate expressed as mm/day, but calculated as a monthly mean value. This is justified because in irrigated areas with frequent applications of irrigation water soil moisture deficits exist only for a short period of time, and on a monthly basis actual evapotranspiration rate, E_a , differs little from potential evapotranspiration rate, E_p . In humid climates the drainage design should probably be based on the demand of the wettest period. During this

period too E_p and E_a will differ little. The designer is then interested in knowing to what extent evapotranspiration can be neglected with respect to rainfall.

In Section 5 the reduction of E_p to arrive at E_a for drying soils is considered. Actual data on evapotranspiration rates are often lacking and hence the design of drainage systems must be based on estimates of E_p obtained from meteorological data. Calculations of E_p based on energy balance considerations are discussed in Section 2 and empirical relationships for the calculation of E_p in Section 3. Methods of measuring evaporation (pan evaporation) and evapotranspiration (lysimeter) are discussed in Section 4. In Sections 5 and 6 some of the consequences of the assumptions made earlier are considered. Finally, Section 7 gives methods of calculating E_p from meteorological data.

19.2 CALCULATION OF POTENTIAL EVAPOTRANSPIRATION BASED ON ENERGY CONSIDERATIONS

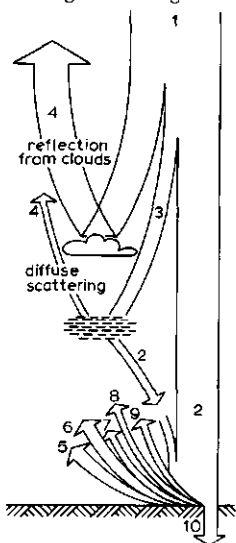
19.2.1 THE ENERGY BALANCE AT THE SOIL SURFACE

Evaporation is one of the components of the water balance, but, through the energy requirement for the phase change of liquid to vapour, evaporation is also one of the terms of the energy balance at the soil surface. The physical requirement underlying the energy balance equation is that all incoming energy be accounted for, because the incoming radiation, the main source of energy, is either reflected, stored as heat, or used for evaporation or some other energy requiring processes, for example photosynthesis. Thus H_o , the net radiation is partitioned as follows

$$H_o = LE + A + G + M \quad (1)$$

where LE is the rate of energy utilization in evapotranspiration ($\text{cal}/\text{cm}^2 \text{ day}$), being the product of the rate of water evapotranspiration and the latent heat of vaporization, i.e. 590 cal/g at about 25°C , A is the energy flux that goes into heating the air, called sensible heat, G is the rate at which heat is stored in the soil, the water, and the vegetation, and M represents miscellaneous energy terms such as photosynthesis and plant respiration. The energy requirement of these miscellaneous processes is usually smaller than the experimental error in measuring the major components and is generally ignored. Only under extreme conditions of a large mass of active vegetation and a relatively low amount of net radiation, as when the sky is overcast, can M account for as much as 5-10% of the net radiation. The net radiation, H_o , is the overall difference between total incoming and total outgoing radiation (both short-wave and long-wave components) (Fig.1). Nearly all of the incoming radiation is of the wave-length range of

0.3-3 microns, and about half of this radiation consists of visible light (with wave-length 0.4-0.7 microns). The radiation emitted by the earth's surface is of greater wave-length than the incoming radiation, i.e. in the wave-length range of 3-50 microns. Short-wave fluxes during the night are negligible and since the long-wave radiation emitted by the earth's surface generally exceeds that received from the sky (i.e. reflected or transmitted from clouds), the net radiation during the night is negative.



- 1 = H_{sh}^{top}
 - 2 = H_{sh}
 - 3 = scattered radiation
 - 4 = reflected radiation from clouds
 - 5 = reflected radiation from surface
 - 6 = H_{10}^n
 - 7 = LE
 - 8 = A
 - 9 = M
 - 10 = G
- $1 = 2 + 4$
 $2 = 5 + 6 + 7 + 8 + 9 + 10$
 $H_o = 2 - 5 = 6 + 7 + 8 + 9 + 10$
 Albedo = $5/2$

Fig.1. Components of the energy balance at the surface

Short-wave radiation

The total incoming short-wave radiation, H_{sh} , can be measured by means of a solarimeter, but these measurements are only being taken at few meteorological stations. Where these data are not available, the total incoming short-wave radiation can be calculated from the incoming short-wave radiation on clear days, i.e. without any cloud cover, or from the extra-terrestrial radiation, H_{sh}^{top} , and the fraction n/N , where n is the actual duration of bright sunshine hours, and N is the maximum possible duration of bright sunshine hours.

The general form of the latter relationship is

$$H_{sh} = H_{sh}^{top} (a + bn/N) \tag{2}$$

where a and b are empirical constants, and H_{sh}^{top} is obtained from Table 3. For the growing season at Rothamsted, England, for example, $a = 0.18$ and $b = 0.55$

KOOPMANS (1969) developed two sets of values, one for temperate climates, $a = 0.20$ and $b = 0.53$, and one for tropical climates, $a = 0.28$ and $b = 0.48$. BAARS (1973) studied the regression equations for a number of locations in Jugoslavia, and found that generally the a -values were higher in summer (0.21-0.30) than in winter (0.14-0.18); the b -values were somewhat lower in summer (0.42-0.47 versus 0.53). Better correlations were obtained when the data for spring and summer were considered separately; then it was found that the a -values were higher in spring than in summer (about 0.28 versus 0.24). For latitudes $\phi < 60^\circ$, the constant a can be approximated by $a = 0.29 \cos \phi$. The calculations presented in Section 7 are made with the values for a and b as suggested by KOOPMANS (1969).

Where the hours of sunshine are not recorded, the ratio n/N , the actual duration of bright sunshine as fraction of the maximum possible duration for a cloudless sky, cannot be calculated. Sometimes this fraction is approximated from cloud cover estimates, but this leads to poor results, particularly in (subtropical) areas where high clouds are present.

Long-wave radiation

The net outgoing long-wave radiation, H_{10}^n , can be calculated according to an empirical formula in which the following assumptions are made (PENMAN, 1948). The emission coefficient for a water surface is taken as 1. The absolute temperature of the radiating surface is assumed to be equal to that of the air. This assumption is valid in temperate climates but probably not in arid or semi-arid tropical regions. The net long-wave radiation can then be calculated from

$$H_{10}^n = 118.10^{-9} (273 + T_2)^4 (0.47 - 0.077 \sqrt{e_2}) (0.2 + 0.8n/N) \quad (3)$$

where T_2 is the temperature at a height of 2 m, and e_2 is the actual vapour pressure (mm Hg) at 2 m. The constants 0.47, 0.077, 0.2 and 0.8 are purely empirical.

BOWEN ratio

BOWEN (1926) recognized that soil heat flux G constitutes only a small fraction of H_0 when soil moisture is not limiting. He thus partitioned H_0 between LE and A terms. The rate of latent heat transfer by water vapour from the field to the atmosphere is proportional to the product of the vapour pressure gradient by the appropriate turbulent transfer coefficient for water vapour. The turbulent transfer coefficient is required because the transport of water vapour (and also of sensible heat) from the field to the atmosphere is affected by the turbulent movement of the air in the atmospheric boundary layer. Turbulent movement of air

does not start immediately in the air layer next to the evaporating surface. A laminar boundary layer of less than 1 mm thick, through which transport occurs by diffusion, exists next to the surface. Similarly, the sensible heat flux, A, is proportional to the product of the temperature gradient and the turbulent transfer coefficient for heat. BOWEN assumed that these transfer coefficients for heat and water vapour are equal. The ratio of the sensible heat transfer to the latent heat transport then becomes

$$\beta = \frac{A}{LE} \approx \gamma \frac{\partial T / \partial z}{\partial e / \partial z} \approx \gamma \frac{T_s - T_z}{e_s - e_z} \quad (4)$$

where γ is the psychrometric constant ($\approx 0.66 \text{ mbar}/^\circ\text{C}$, or $0.485 \text{ mm Hg}/^\circ\text{C}$), the subscript s indicating a measurement at the surface, and the subscript z a measurement at height z.

The validity of the assumption that the transfer coefficients for sensible and latent heat are equal has been the subject of considerable disagreement among microclimatologists interested in evapotranspiration studies. However, it can be shown that the error in LE is considerably smaller than any error in β if the assumption of identity between the two coefficients is in error.

The ratio β is called the BOWEN ratio; β is rather small when most of the energy is used for evaporation. This is the situation when the field is moist and the relative humidity gradient tends to be large and the temperature gradient small. Conversely when the field is dry the BOWEN ratio can be large because of a small gradient in vapour pressure and a rather large temperature gradient. Under extremely dry conditions LE may tend to zero and β therefore to infinity.

19.2.2 EVAPORATION FROM A FREE WATER SURFACE

Application of the energy balance equation for the calculation of evaporation requires measurements of the temperature of the evaporating surface. The temperature of this surface is rather difficult to estimate in case of a free water surface and virtually impossible for vegetation. The vapour pressure at a free water surface is the vapour pressure at saturation for the existing temperature at the surface ($e_s = e_{s \text{ sat}}$). So when the surface temperature is known the corresponding vapour pressure can be obtained from Table 1.

PENMAN (1948) was the first to propose a semi-empirical method for calculating the evaporation from a water surface. His method is derived from a combination of the DALTON equation and the energy balance approach. This obviates the need

to measure surface temperature, but necessitates the introduction of approximations, in particular with regard to the turbulent transfer coefficient. The general form of the DALTON equation for a free water surface is

$$LE = f(u) (e_{s \text{ sat}} - e_z) \quad (5)$$

where $f(u)$ is a function of the wind speed above the surface, $e_{s \text{ sat}}$ is the saturation vapour pressure at the temperature of the surface, and e_z is the vapour pressure in the air at height z .

Since the temperature and vapour pressure at the surface are not easily measurable, PENMAN introduced a "standard" value of LE, namely

$$LE_x = f(u) (e_{z \text{ sat}} - e_z) \quad (6)$$

where $e_{z \text{ sat}}$ signifies the saturation vapour pressure of the evaporating surface for the condition that the surface temperature equals the air temperature, T_z , which is easily determined. The wind function, $f(u)$, according to PENMAN, equals

$$f(u) = (0.5 + 0.54 u_2)0.35 \quad (7)$$

where u_2 is the wind velocity in m/sec at a height of 2 m. The vapour pressure, e_z , should also be determined at that height (Sect.7). A major limitation of the method, perhaps preventing its wide application, is the difficulty of evaluating the wind function, $f(u) = a + bu_z$, since the empirical "constants" a and b , as used in Eq.7, depend on the nature of the surface. For accurate work this requires some years of comparison with some standard method of evaluating evaporation, such as lysimetry or energy balance measurements.

PENMAN combines the ratio LE_x/LE (Eqs.5 and 6) with the energy balance (Eq.1) and the BOWEN ratio (Eq.4), to write LE and A in terms of H_o , G, and β only, neglecting the miscellaneous energy term M in Eq.1 as follows

$$LE = (H_o - G)/(1 + \beta) \quad (8)$$

and

$$A = \beta(H_o - G)/(1 + \beta) \quad (9)$$

PENMAN assumed that $G = 0$, so Eq.8 could be simplified to

$$LE = H_o/(1 + \beta) \quad (10)$$

When Eq.6 is divided by Eq.5 we have, after rearranging terms

$$\frac{LE_x}{LE} = 1 - \frac{e_{s \text{ sat } z} - e_z}{e_{s \text{ sat } z} - e_z} \quad (11)$$

From Eq.10 by writing

$$\beta \text{ as } \gamma \left(\frac{T_s - T_z}{e_{s \text{ sat } z} - e_z} \right) \text{ we obtain}$$

$$LE = \frac{H_o}{1 + \gamma \left(\frac{T_s - T_z}{e_{s \text{ sat } z} - e_z} \right)} = \frac{H_o}{1 + \gamma \left(\frac{T_s - T_z}{e_{s \text{ sat } z} - e_z} \right) \left(\frac{e_{s \text{ sat } z} - e_z}{e_{s \text{ sat } z} - e_z} \right)} \quad (12)$$

$$\text{Within a limited temperature range } \Delta = \frac{e_{s \text{ sat } z} - e_z}{T_s - T_z} \quad (13)$$

where Δ is the slope of the saturation vapour pressure - temperature curve. Equation 12 can be written as

$$LE = \frac{H_o}{1 + \gamma \left(\frac{1}{\Delta} \right) \left(1 - \frac{LE_x}{LE} \right)} \quad (14)$$

Rearranging of terms gives

$$LE \left(1 + \frac{\Delta}{\gamma} \right) = \frac{\Delta}{\gamma} H_o + LE_x \quad (15)$$

which leads to the PENMAN equation

$$LE = \frac{\left(\frac{\Delta}{\gamma} \right) H_o + LE_x}{1 + \left(\frac{\Delta}{\gamma} \right)} \quad (16)$$

This equation enables us to calculate the evaporation from a free water surface from measurements of the net radiation, the temperature, vapour pressure, and wind velocity at one particular level above the surface. The application, therefore, is limited to those parts of the world where these meteorological quantities are determined on a routine basis.

19.2.3 POTENTIAL EVAPOTRANSPIRATION BY CROPS

In Eq.16, H_o refers to the net radiation over a free water surface. The question arises whether the equation could also be used directly over a vegetated surface if the appropriate net radiation term were measured, or if a value for the

surface albedo were used. The albedo (i.e. reflectivity averaged over all relevant wave-lengths) for plant surfaces is generally much greater than the 0.03 to 0.06 for water surfaces, and varies with the degree of soil coverage, colour, and hence with the maturity of the crop. Values for cotton range from 0.18 at the beginning of the growing season to 0.27 for a crop of 115 cm (FRITSCHEN, 1967). HOUNAM (1971) proposes an albedo value of 0.25 for a green crop to adjust other radiation measurements or estimates if direct observations of crop H_0 cannot be made. According to PENMAN (1956) the potential evapotranspiration rate is determined by the prevailing weather conditions and, for a crop completely covering the soil surface and never short of water, the rate is about the same, irrespective of plant or soil type. A corollary of this thesis is that evapotranspiration from a short green cover cannot exceed the evaporation from an open water surface exposed to the same weather. Recent research into aerodynamic roughness factors suggests some significant variations between crop types (SLATYER and McILROY, 1961, RIJTEMA, 1966, and ROSENBERG, HART, and BROWN, 1968), and it has been well established that crop evapotranspiration can exceed free water loss, particularly when there is appreciable advective transfer of heat (TANNER and PELTON, 1960, PRUITT, 1963). Advection is the horizontal transfer of heat which can contribute energy for evapotranspiration but is not considered in the energy balance equation where only vertical fluxes are included. When advection occurs, sensible heat is transferred from the air to the surface, and the BOWEN ratio will be negative during that portion of the day. The influence of advection can extend to a distance of about 1 km from the boundary between the heated non-evaporating arid surface and the irrigated area.

In practice, potential evapotranspiration rate is often estimated from the measured or calculated evaporation rate from a free water surface. A simple correlation between the two is then assumed to exist

$$E_p = f E \quad (17)$$

where E_p is the potential evapotranspiration rate.

The empirical conversion factor f depends on the reflectivity of the crop, on the season, the climate, and the size of the cropped area. The table below (VAN DER MOLEN, 1971) can be used to give a first approximation.

Values of the factor f, in $E_p = f E$						
Crop	Humid			Arid or semi-arid		
	temperate		tropical	large area		small (less than 1ha)
	winter	summer		winter	summer	summer
Wet, after rain or irrigation	0.9	1.0	1.0	1.0	1.2	1.5
Short grass	0.7	0.8	0.8	0.8	1.0	1.2
Tall crop (wheat, sugar cane)	0.8	1.0	1.0	1.0	1.2	1.5
Rice	1.0	1.0	1.2	1.0	1.3	1.6

In this table differences in the multiplier f resulting from physiological variation of the crop during the growing season are not considered.

19.3 CALCULATION OF POTENTIAL EVAPOTRANSPIRATION BASED ON EMPIRICAL RELATIONSHIPS

Many empirical methods have been developed for estimating evaporation from open water surfaces or evapotranspiration from vegetation. Most of these have employed commonly measured meteorological elements to find a correlation with evaporation E, or evapotranspiration E_p , valid for a limited area. The use of these methods is quite legitimate in such an area provided the period considered is not less than about 20 days. It is hazardous, however, to use a method in another climate than that for which it was derived, or to make estimates over short periods.

Methods based on temperature and radiation

In Section 7 two methods of calculating E_p , both based on temperature and some index of radiation, are presented. These are methods developed by TURC (1954), and JENSEN and HAISE (1963). Generally it is found that both formulas tend to underestimate E_p during spring and overestimate it during summer. This results from the fact that temperature is given too much weight and radiation not enough in these formulas. If radiation data are available PENMAN's formula is therefore to be preferred.

Methods based on temperature and daylength

Also presented in Section 7 are two formulas based merely on temperature and daylength; these were developed by BLANEY and CRIDDLE (1950) and THORNTHWAITE (1948). There is evidence that THORNTHWAITE's value can serve a useful purpose in determining water requirements of crops over months and seasons, when mutually high correlations exist between mean temperature, net radiation, and evapotranspiration. For

Evapotranspiration

shorter periods the method fails principally because mean temperature is not a suitable physical measure of the energy either available for or used in evapotranspiration (HOUNAM, 1971). Empirical relationships based solely on temperature have no practical value as short-term predictors, particularly in a climate of marked seasonal contrast in atmospheric moisture content (STERN and FITZPATRICK, 1965). The BLANEY-CRIDDLE procedure was originally developed for estimating seasonal "consumptive use" (i.e. comparable to seasonal potential evapotranspiration because of the underlying assumption that water supply to the growing crop never becomes limiting). Other assumptions are that the length of the growing season is an index of production and water use by continuously growing crops, and that fertility and water storage of the soil do not differ significantly among areas to be compared. When the method was used for the prediction of evapotranspiration for consecutive short periods throughout the growing season, it became necessary to modify the crop coefficient, C_1^{BC} , in Eq.39 (SCS, 1967). One modification requires the use of climatic coefficients that are directly related to the mean air temperature for each of the short periods which constitute the growing season

$$C_T = 0.0312 T + 0.24 \quad (18)$$

where T is the mean air temperature ($^{\circ}$ C). The other modification requires the use of coefficients which reflect the influence of the crop growth stages on E_p . These coefficients should ideally be obtained in lysimeter studies. The value of the coefficients might to some extent be influenced by factors other than the plant characteristics, and hence it is not expected that the coefficients can be used universally. The method is widely used, particularly in the U.S.A., where through the use of the appropriate crop coefficients an estimate of actual rather than potential evapotranspiration can be obtained when the coefficients are based on correlation with the existing irrigation practice. As was stated earlier, estimates obtained by this method may be realistic only for localities and periods for which the coefficients used were developed. Several comparisons between the various empirical approaches are available in literature. VAN BAVEL and WILSON (1952) tested the THORNTHWAITE, PENMAN, and BLANEY-CRIDDLE methods against evaporation pans and found fair agreement. HALKIAS et al. (1955) tested the same methods against atmometer evaporation (Sect.4) and gravimetric soil moisture measurement techniques, and suggested that only soil moisture samples give reliable data on extraction. HANKS et al. (1971) found that potential evapotranspiration from a lysimeter exceeded the evaporation from a wet surface (PENMAN) by 30% under the advective conditions of Northern Utah. A comparison of the methods presented in this chapter is given in Section 7 for an experimental site in Venezuela.

19.4 MEASURING EVAPORATION AND EVAPOTRANSPIRATION

19.4.1 THE WATER BALANCE METHOD

The water balance, referred to in the introduction, may be thought of as a direct means of measuring potential evapotranspiration, but unfortunately, the degree of accuracy is rather low except in special cases, such as when lysimeters are used (Sect.4.2).

The water balance may be written as

$$\Delta W = P + I - R - D - E \quad (19)$$

where ΔW is the change in water content in the rootzone during a certain period, P is the precipitation in that period, I the irrigation, R the runoff, D the deep drainage from the rootzone, and E is the combined loss due to transpiration and evaporation. This is an integral form of the water balance, with the various items totalled over a certain period of time. The water balance can also be expressed in differential form, referring to the time rates of the processes, as the rate at which water flows downwards through the lower boundary of the rootzone, and the evapotranspiration rate. In differential form, the left hand side of the equation becomes the rate of change of the water content in the rootzone. In irrigated areas where runoff is negligible, the water balance for periods between irrigations (or rainfall) can be reduced to

$$-\Delta W = D + E \quad (20)$$

Soil moisture depletion

It is common practice in irrigation to schedule irrigations according to the degree of deficiency of the total water content of the rootzone. Available water is then considered to be that held between field capacity and permanent wilting point. However, with most crops it is not wise to use all of the so-called available water between irrigations because yields decrease when soil water is allowed to approach the wilting point. Often irrigations are scheduled when 50% of the available water has been used. The usual procedure for determining field capacity is by periodic sampling of a wetted soil that is kept covered to prevent evaporation. Plants, of course, transpire water between an irrigation and the time that the field capacity condition is attained (e.g.6 days). This water must be taken as part of the available water. Deep drainage also occurs and this water cannot be regarded as being available. In studies of the water balance as given in Eq.20, depletion of soil water, as calculated from water content changes

found from periodic gravimetric sampling of the rootzone, has often been equated with losses due to evapotranspiration. Very few attempts have been made to partition depletion into evapotranspiration and downward or upward flow of water through the boundary of the soil layer under study because of insufficient data. VAN BAVEL et al. (1968), however, measured evaporation losses independently in a study of the water balance of a soil layer. By combining these direct evaporation measurements with known depletion figures, they obtained an estimate of the downward or upward flux of water. Their data demonstrated that when a bare soil is irrigated the depletion rate is always more than the true evaporation rate. When a sorghum crop with a normally developed root system was present, the initial depletion rates were also greater than the measured evaporation rates, but later, as water moved upward into the rootzone the depletion rate was an underestimate of the evaporation rate.

BLACK et al. (1969) successfully predicted the evaporation from a bare sand surface for an entire season by using only rainfall and irrigation inputs. They assumed that each time it rained or the soil was irrigated the evaporation rate declined as the square root of time and that the cumulative evaporation increased proportionally to the square root of time. The drainage rate was found to be an exponential function of water storage. This procedure may not be generally applicable since the relation between the square root of time and evaporation is applicable for a soil which is wet to an infinite depth. For a finite depth of wetting the rate drops off more rapidly; this occurs also with heavier textured soils (KIJNE, 1972).

Soil moisture depletion in the presence of a plant cover

Evaporation from soil under plant canopies or between rows in the case of row crops cannot be determined by covering the soil surface and subtracting measured transpiration from the sum of evaporation and transpiration from the uncovered plot. This is because much of the energy which strikes the covered soil surface is reflected and results in increased transpiration. BLACK et al. (1970) measured the energy that is available at the soil surface under snap beans and estimated the ability of the soil to supply water from the surface water content. They then assumed that the evaporation was limited by the energy or by the soil-water flux, whichever was smaller. Independent calculations of transpiration were made based on leaf-stomatal-resistance values and meteorological measurements within the canopy. The two independently obtained values of evaporation were in good agreement. BRUN et al. (1972) reported a similar study on soybean and sorghum in which

again fair agreement was obtained between the two independent estimates of the evapotranspiration, except under conditions of high evaporative demand when the sum of calculated soil evaporation and transpiration was less than the lysimetric estimates of evapotranspiration.

The drainage component of the water balance under ordinary field conditions can be substantial, as was shown, for example, by the results of the study by BLACK et al. (1970). In their study evapotranspiration accounted for 170 mm and drainage for 180 mm of the seasonal precipitation of 320 mm and the 30 mm decrease in water storage. As changes in soil moisture content are generally only a small fraction of the total moisture content, the accuracy of determining such changes will be much lower than that of the soil moisture measurement itself. Even with considerable effort and elaborate sampling procedures and equipment, good results can rarely be expected over periods of less than a week (Chap.23, Vol.III). The upward movement of water from the water table should also be considered in studies of the water balance. Some uncertainty still exists regarding the distance which water can rise under the influence of hydraulic potential gradients, but evidence by GARDNER (1965) suggests that rises from a 90 cm water table in sandy loam are quite feasible. The rate of water extraction depends on the rooting characteristics of the plant, the soil type, the water potential in the rootzone, and the depth of the water table (Chap.5, Vol.I).

19.4.2 THE USE OF EVAPORIMETERS

Lysimeters

The most direct way of measuring the components of the water balance as given in Eqs. 19 or 20 is by means of lysimeters. These are large containers of soil set in natural surroundings with the least possible discontinuity between the crop on the lysimeter and that in the surrounding field (Fig.2A). The method by which the lysimeter is filled with soil can be very important. In some instances, for example, with a more or less amorphous sandy profile, back fill is satisfactory, particularly if care is taken to return unit layers of soil in the reverse order of extraction. However, in heavier soils, particularly where there are discontinuities in depth, it is generally necessary to carry out a "monolith" fill operation in which the undisturbed column of soil is fitted into the container (HOUNAM, 1971). The effective use of lysimeters is limited to situations in which the vegetation community under study can be simulated within the lysimeter itself, without, for instance, any restriction of root development. Lysimeters which can be weighed either by means of pressure transducers or because they float in water

or some other liquid provide a means of accurately determining the water loss due to evapotranspiration, provided the temperature sensitivity of the weighing system is properly taken into consideration. A reliable measurement of the components of the water balance is only obtained if the water potential profile in the lysimeter is the same as in the surrounding field. This condition can be satisfied if the lysimeter is provided with a drainage system and a system to maintain the water potential at the bottom of the soil in the lysimeter at the same level as the water potential in the adjacent field. When, however, the object is to determine the maximum evapotranspiration rate, the moisture condition in the soil column is not critical as long as root growth is normal. These lysimeters should then be irrigated frequently, e.g. every 4 or 5 days, unless rainfall intervenes. A fairly reliable estimate of E_p for periods of a week or more should then be possible. The use of constant water table lysimeters for the measurement of E_p for short periods is questionable, since under conditions of high evaporative demand, the movement of water from the water table into the rootzone may not always be rapid enough to equal the potential evapotranspiration rate.

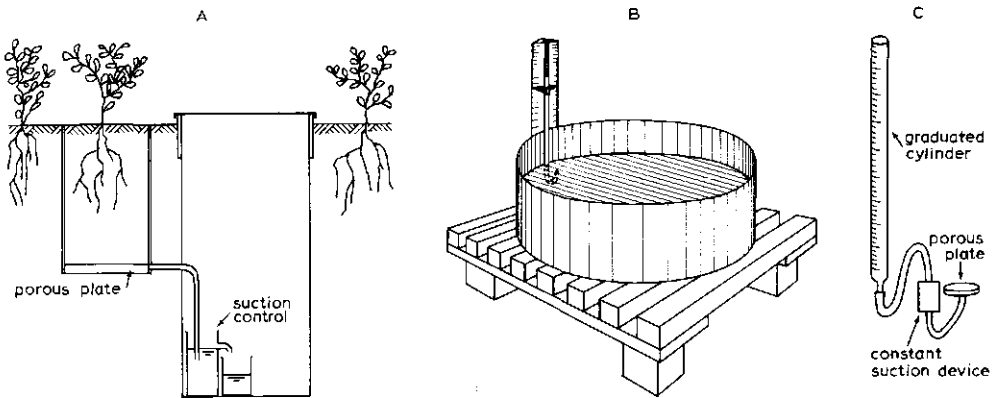


Fig.2. Evaporimeters. A: non weighable lysimeter with suction control at the bottom. B: evaporation pan. C: atmometer

Pan evaporation

The evaporation from a free water surface of an open pan (Fig.2B) is widely used as an indicator of the evaporative demand of the atmosphere. In the operation of the pan, runoff and drainage are prevented, and evaporation is given by the change in water level in the pan after allowance is made for precipitation. Pan evaporation depends on the dimensions and exposure of the pan and the materials

from which it has been constructed, as well as on all the meteorological conditions. Because of their ease of operation, pans are widely used throughout the world, but their limitations have not always been clearly understood. One factor to be considered is standardization so that comparisons can be carried out between results obtained at different locations. The Class A pan of the U.S. Weather Bureau (4 feet in diameter and 10 inches high) is most widely used as the standard pan. Due to absorption of radiation through the pan wall and transfer of sensible heat between the air and the pan wall, the above-ground pan receives an additional amount of energy, which results in higher evaporation rates than are calculated from meteorological data. Pans are even more susceptible to advection than are crops, as has been shown by PRUITT et al.(1972). Sunken pans might then be expected to give more reliable results, but heat exchange between pan wall and surrounding soil, and surface roughness effects limit the accuracy of results obtained with them. Empirical correlations are required between the measured evaporation rate from the pan and either E_p or E_a (under standard irrigation practice), whatever may be required.

Atmometers

Reasonable correlations have been found between the evapotranspiration from irrigated crops and the evaporation from atmometers. Atmometers are instruments with a porous surface connected to a supply of water in such a way that evaporation occurs from the porous surface (Fig.2C). A common atmometer is the PICHE atmometer, made from a flat, horizontal disk of wetted blotting paper with both sides exposed to the air; another is the black and white spherical porous ceramic atmometer, and a third is the BELLANI blackplate atmometer, which consists of a flat, black porous ceramic plate as the upper face of a non-porous hemisphere. The net radiation flux density per unit evaporating surface of an exposed BELLANI is of the same magnitude as a crop surface whereas that of the other two types is about one-half because evaporation occurs at both the upper and lower surfaces. Evaporation from atmometers is also affected by heat conduction through the water from the supply system. Furthermore, the transfer of sensible heat from the air is much greater with atmometers than with vegetation because the atmometer is usually placed at some height above the crop. Nevertheless, in many instances satisfactory correlations have been found between the evaporation from the atmometer and potential evapotranspiration from crops. For example, KORVEN and WILCOX (1965) based the irrigation schedule of an irrigated orchard on the evaporation from a BELLANI atmometer. It has also been found that the difference in evaporation between black and white spherical atmometers was highly correlated with potential

evapotranspiration. The difference between similarly exposed black and white units depends mainly on radiation since the convective transfer to each is about the same. Provided the condition of the evaporating surface is properly maintained, atmometers may well be used as standards of reference for different times at any one location. Comparison of evaporative conditions between sites by means of atmometers seems less likely to be successful because of the sensitivity of these instruments to convective heat exchange and hence to the influence from the surrounding fields.

19.5 ADJUSTMENT OF EVAPOTRANSPIRATION FOR DRYING SOILS

The difference between potential and actual evapotranspiration may be small for soil surfaces, which are well supplied with water and completely covered by vegetation, especially for those periods of time considered in drainage design. For short periods, however, and also under conditions of limited water supply or sparse vegetation, the increase in resistance to diffusion of water vapour in the soil and through the stomata of plant leaves, developing when the soil dries out, has to be considered. There is considerable evidence that the potential evaporation rate (evaporative demand of the atmosphere) exerts a definite influence on the relationship between the fraction of actual over potential evapotranspiration in its dependence on the availability of soil moisture expressed as either soil moisture content or matric potential (e.g. DENMEAD and SHAW, 1962; BAHRANI and TAYLOR, 1961). An example of the dependence of the relative evapotranspiration rate (E_a/E_p) on the matric potential of the soil water is given in Fig.3. The moisture-retaining capacity of the soil and the extent of the root system also affect the relationship between evapotranspiration and soil moisture. Several simplified models have been advocated for the relation between E_a/E_p and soil moisture content or matric potential. The general form of this relation for the drying of soil is

$$E_a = kE_p \quad (21)$$

in which k is a function of soil moisture content or matric potential. Some of the k -moisture content relations are depicted in Fig.4. The simplest is given by $k = 1$ for moisture contents greater than that corresponding to the permanent wilting point (W_{wp}). ($W > W_{wp}$), and $k = 0$ for $W = W_{wp}$ (Curve a, Fig.4). Other relationships are given by curves b and c (Fig.4) in which k decreases linearly with the fraction of soil moisture remaining between field capacity (W_{fc}) and permanent wilting point

$$k = \frac{W - W_{wp}}{W_{fc} - W_{wp}} \quad (22)$$

and between field capacity and oven-dry soil (W_o)

$$k' = \frac{W - W_o}{W_{fc} - W_o} \quad (23)$$

Curves d_1 and d_2 in Fig.4 represent curvilinear relationships. From Eqs.21, 22, and 23 we have

$$k = \frac{E_a}{E_p} = \frac{W_a}{W_p} \quad (24)$$

where W_a is the amount of moisture remaining in the soil and W_p the maximum amount available in the soil for evapotranspiration. In other words, the actual evapotranspiration is a function of the amount of moisture remaining in the soil

$$E_a = cW_a \quad (25)$$

For the drying of soil, without additions resulting from irrigation, rainfall, or capillary rise from the groundwater table

$$E_a = - \frac{dW_a}{dt} \quad (26)$$

and, therefore,

$$cW_a = - \frac{dW_a}{dt} \quad (27)$$

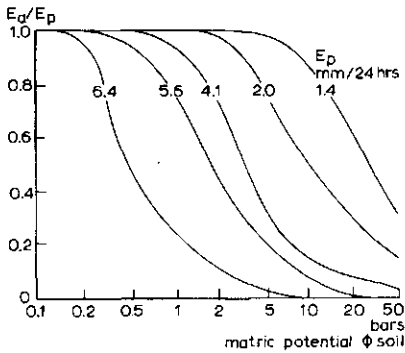


Fig.3. Actual dependence of relative evapotranspiration rate on ϕ_{soil}

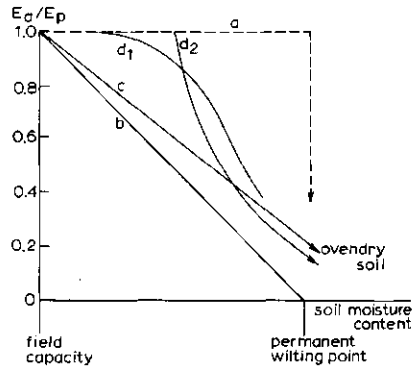


Fig.4. Relationship between relative evapotranspiration rate and soil moisture content

Evapotranspiration

For the conditions that at the start of drying, i.e. $t = 0$, $W_a = W_p$, $E_a = E_p$, we have from Eq.27

$$W_a = W_p e^{-E_p t / W_p} \tag{28}$$

in which t = time in days since the initiation of drying. The equation was first derived by THORNTHWAITE and MATHER (1955). Figure 5 shows a plot of W_a as function of time for $W_p = 150$ and $E_p = 5$ mm/day.

NORERO et al.(1972) have proposed a mathematical expression for a curvilinear relationship which appears to fit well with the presently available experimental data. The decrease in relative evapotranspiration rate (E_a/E_p) as function of time can be calculated from their expression. This is plotted in Fig.6, for data obtained with corn (DENMEAD and SHAW, 1962; $E_p = 5.6$ mm/day, $W_p = 85$ mm), together with the E_a/E_p -time relation for the same data calculated from the THORNTHWAITE and MATHER equation (Eq.28). The two curves are rather different for periods of less than 14 days; thereafter, they coincide reasonably well.

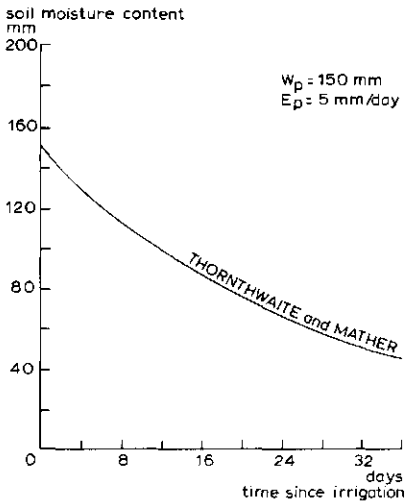


Fig.5. Decrease of soil moisture content as function of time

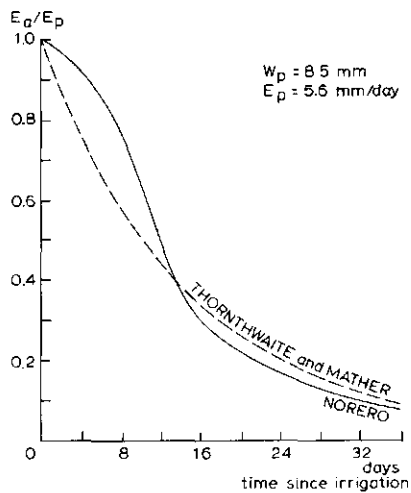


Fig.6. Relative evapotranspiration as function of time (after data from DENMEAD and SHAW, 1962)

From the work by NORERO et al.(1972), it may be concluded that for practical purposes the assumption that E_a/E_p is unity when soil moisture storage is at a maximum and that the ratio is zero when soil moisture storage is zero, is not unreasonable, especially with a fairly high evaporative demand. When the demand is low, assuming the ratio to be one, while the average ratio between irrigation applications was only 0.8, would imply an inaccuracy of 20% in the estimate of stored water. This, in turn, could mean a considerable contribution to the groundwater tables over an entire season, if irrigation applications were based on the estimated stored water.

Stage of growth

The stage of growth is also an important factor in the water requirement. Evapotranspiration increases gradually from planting time to maturity and thereafter it decreases gradually. The gradual increase can be explained on the basis of percent cover, which is particularly important for row crops that start out from a bare soil and approach 100% cover at maturity. Evaporation for most bare soils decreases rapidly 1 or 2 days after an irrigation or a rain. Under the same conditions, transpiration may be maintained at the same level for several weeks. Consequently evapotranspiration would increase as the percent cover increases. An additional effect on plant cover is associated with the reflection coefficient for incoming radiation, which is lower for bare soil (especially when wet) than for a dense crop. Low reflection implies that more energy will be absorbed and hence is available for evaporation and heating. Based on reflection alone evapotranspiration would be expected to decrease as the percent cover increases. However, this effect is far less important than the first, so that a gradual increase in evapotranspiration is associated with an increase of soil cover. The gradual decrease with stage of growth at the end of the growing season probably results from plant physiological factors since it has been found that the decrease of evapotranspiration from cotton occurs under conditions of continued irrigation as well as when the water application is cut down after maturity, as is usual in irrigation practice.

Evaporation from bare soil

Evaporation takes place at nearly the same rate from saturated soil, as may occur in an irrigated area as from a free water surface, except in the leading edge where advected heat may result in higher losses. Evaporation from unsaturated soil decreases rapidly as the soil dries out near the surface. Vapour transfer across a desiccated surface layer is slow and its rate is determined by temperature gradients within the surface soil (Chap.5, Vol.I).

19.6 AREAL VARIATION AND FREQUENCY OF OCCURRENCE OF EVAPOTRANSPIRATION

19.6.1 EXTENSION FROM POINT TO AREAL DATA

Most of the methods discussed in the previous sections give estimates of evapotranspiration at a single observational site, or over a relatively small area in the neighbourhood of the sensor. The errors in these estimates are of the order of 15-30% depending on the length of the period for which the estimate is made. It is necessary to consider the possibility of integrating a network of point values to give a representative evapotranspiration for an area (HOUNAM, 1971). Methods developed for the integration of point values of rainfall (Chap.18, Vol.III) may be used for this purpose. The most appropriate method for integrating point observations will depend to a considerable extent on the nature of the variation in evapotranspiration across the area. Spatial variations are due to variations in radiation load on the vegetation, which may be the result of climatological variation on a horizontal surface (i.e. temporal variation in cloud cover), and variation due to slope, orientation, and elevation of the terrain. Additional variations result from differences in soil moisture availability and transpiration characteristics of the vegetation. The reflectivity of a surface has so far been treated as a constant but it actually varies with the wave-length and, to a certain extent, the angle of incidence. The albedo of a wet surface varies between 0.02 (at zenith sun for the direct beam) and nearly 0.60 at an angle of 85° (LIST, 1966). Since the albedo is only 0.06 at 60° , a constant value of 0.03-0.05 can be used for most computations. Angle of incidence is far less important in the case of plants, because of variations in leaf shape and orientation, but temporal variations in the growing season and variations amongst crops are considerably greater than the small change quoted for water. Using the PENMAN equation, HOUNAM (1971) has calculated, that evaporation increases by about 25% if the surface albedo is varied from 0.30 to 0.20 (for air temperature 17°C , relative humidity, $h=0.63$, 50% cloud cover). In view of the significance of albedo in the evaporation process, its possible spatial and temporal variations (e.g. due to crop rotations or the presence of mulches) need to be considered. Apparently wide variation in evapotranspiration is to be expected, particularly in areas of diversified terrain and land-use, but many of the spatial and temporal variations have not been quantified so far.

19.6.2 FREQUENCY DISTRIBUTION OF EVAPOTRANSPIRATION

Because of lack of information, little attention has been given to frequency distributions of peak water use. In only a few locations in the world have evapotranspiration data been collected for a single crop over a period of time that is

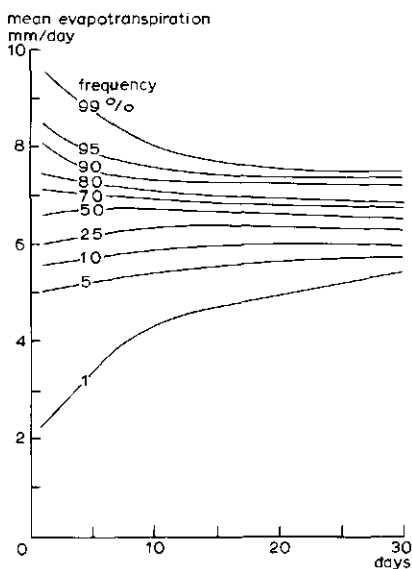


Fig.7. Frequency distribution curves for periods of daily evapotranspiration and means for various lengths of consecutive-day periods up to 30 days in length

adequate for a worthwhile frequency-distribution analysis. One such record comes from work with the large weighable lysimeter at Davis, California, which has been continuously cropped to a grass cover since June 1959 (PRUITT et al., 1972). The actual peak evapotranspiration rates experienced at Davis are unique for the climate involved, but the general characteristics of the frequency distribution may be rather widely applicable in the Mediterranean-type hotter climates in many parts of the world. Cumulative frequency distribution curves for periods of daily evapotranspiration and means for various lengths of consecutive-day periods illustrates why long-term mean weather data (e.g. monthly values) are not sufficient for predicting evapotranspiration demand in the design of irrigation and drainage projects and/or farm systems. For example, even considering deep-rooted crops grown in fertile soils, where there might be 15-23 cm of readily available soil moisture following irrigation, the data from Davis (Fig.7) show that a design accounting for 7.4 mm per day is required if the system is to cover 95% of the cases, rather than the 10-year monthly mean for the same period of peak demand of around 6.6 mm per day. For shallower-rooted crops, or crops otherwise benefiting from more frequent irrigation, the design value for Davis conditions would run as high as 8.1 mm per day to handle 95% of the cases, if around 2.5 cm of stored water could be considered readily available. At the 99% level, the design values would be considerably higher, but of course, it will seldom be necessary to design

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for peak 1-day values at the 99% level or even at the 95% level. If one could economically accept the capacity to meet the peak 5-day demand all but 30% of the time (i.e. 70% level), the design requirement could be reduced, but it still would be 7 mm/day, i.e. higher than the 10-year monthly mean of 6.6 mm per day. The designer must consider the effects of not fully replenishing the soil water every 5 days in selecting the desired probability level of evapotranspiration that the system will meet. Obviously, the irrigation design has repercussions for the desired capacity of the drainage system (Chap.10, Vol.II).

19.7 EXAMPLES OF CALCULATING POTENTIAL EVAPOTRANSPIRATION¹

Five methods have been mentioned in the previous sections for the calculation of potential evapotranspiration. In table below the meteorological data required for each of these is indicated.

Meteorological data required for various methods of calculating potential evapotranspiration						
Method	T	n ₂	u	or u ₁₀	Relative humidity h or e ₂	Rainfall P
Penman	x	x		x	x	
Turc	x	x			x	x
Jensen-Haise	x	x				
Blaney-Criddle	x					
Thornthwaite	x					

Before the methods of calculation are presented, with the appropriate tables and nomographs, it should be indicated which value should be used of those meteorological quantities that vary throughout the day and are measured more than once.

- The temperature to be used in the equations and for the determination of the saturation vapour pressure (from tables) is usually an average value. The average temperature for a day can be obtained from hourly observations, if these are available, or from the temperature readings at 8,14, and 19, or 1,7,13, and 19 hours, whatever is practicable. The daily mean value can also be calculated from the maximum and minimum temperature. The results do not differ greatly.

- The actual vapour pressure at a height of 2 m is obtained from the wet and dry bulb temperatures at that height. The actual vapour pressure can also be calculated from measurements of temperature and relative humidity since $e_2 = h_{sat}$.

¹ The collaboration with Dr C.Baars, Dept.of Irrigation and Civil Engineering, University of Agriculture, Wageningen, in the preparation of this section is gratefully acknowledged.

where h is the relative humidity. The information on the temperature and relative humidity should then be available for the same time.

- The wind velocity is required for PENMAN's formula at a height of 2 m. Wind velocities at other heights (u_z) can be converted to a value for 2 m through the following relationship

$$u_2 = u_z \frac{\log \frac{200}{2}}{\log \frac{100z}{2}} \quad (29)$$

In the following sections E and E_p will now be calculated for July, for a temperate climate location at 42.5°N , according to the above-mentioned methods.

The meteorological data are as follows

Average air temperature at 2 m (T_2)	= 23.1°C
Average actual vapour pressure at 2 m (e_2)	= 12.9 mm Hg
Average wind velocity at 10 m (u_{10})	= 2.19 m/s
Average duration of bright sunshine (n)	= 9.74 hrs/day

For July at 42.5°N we find from Table 9 (Annex I) that the maximum possible duration of bright sunshine hours equals 14.96 hours. The relative sunshine duration n/N is then $9.74/14.96 = 0.65$.

We find from Table 3 (Annex I) for the H_{sh}^{top} value $961 \text{ cal cm}^{-2}\text{day}^{-1}$.

19.7.1 FORMULA DEVELOPED BY PENMAN

The basic formula for the calculation of evaporation from a free water surface is as follows

$$LE = \frac{\left(\frac{\Delta}{\gamma}\right)H_o + LE_x}{1 + \left(\frac{\Delta}{\gamma}\right)} \quad (30)$$

where

- E = evaporation from a free water surface (mm day^{-1} ; 1 day = 24 hours)
- H_o = net radiation ($\text{cal cm}^{-2} \text{ day}^{-1}$)
- E_x = isothermal evaporation (mm day^{-1})
- Δ = slope of the temperature-vapour pressure relationship at temperature T ($\text{mm Hg } ^\circ\text{C}^{-1}$)
- γ = psychrometric constant ($=0.485 \text{ mm Hg } ^\circ\text{C}^{-1}$)
- L = latent heat of evaporation of 0.1 cm^3 ($=59 \text{ cal}$).

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The aerodynamic term, E_x , can be calculated according to PENMAN from the following equation

$$E_x = 0.35(0.5 + 0.54 u_2)(e_{sat} - e_2) \quad (31)$$

where

u_2 = wind velocity at 2 m ($m \text{ sec}^{-1}$)

e_{sat} = saturated vapour pressure (mm Hg)

e_2 = actual vapour pressure at 2 m (mm Hg)

Calculation by means of tables

Equation 16 can be written in terms of the variables and constants discussed in Section 2.1

$$E = \frac{\frac{\Delta}{59} 0.94(a+bn/N) H_{sh}^{top} - 118.10^{-9}(273+T_2)^4 (0.47-0.077 \sqrt{e_2})(0.2+0.8n/N)}{\Delta + 0.485} + \frac{0.485 \times 0.35(0.5 + 0.54 u_2)(e_{sat} - e_2)}{\Delta + 0.485} \quad (32)$$

Tables have been prepared by WESSELING (1960) and others for different sections of Eq.32. BAARS (1973) extended the tables of WESSELING, and these tables have been used in this paper (see Annex I).

In Table 1, the value of Δ is given as a function of temperature.

In Table 2, values are given for $(a+bn/N)$ with the constants which KOOPMANS derived for temperate and tropical climates. The value of $(a+bn/N)$ is given as a function of n/N .

In Table 3, the extra-terrestrial radiation (H_{sh}^{top} or Angot-value) is given as a function of latitude, for periods of 10 days and for months of the year. Note that the table gives values for the Northern and Southern Hemispheres.

In Table 4, values of $118.10^{-9}(273+T_2)^4$ are listed as a function of temperature.

In Table 5, values of $0.47-0.077 \sqrt{e_2}$ are listed as a function of the actual vapour pressure at 2 m, e_2 .

In Table 6, values of $0.2+0.8 n/N$ are listed as a function of n/N .

In Table 7, values are listed of $0.485 \times 0.35(0.5+0.54u)$ for u_2 (m/sec) measured at 2 m and for u_{10} (m/sec) as measured at 10 m. It is, therefore, not necessary

to convert readings of wind velocity obtained at 10 m to a value for 2 m through Eq.29 since the appropriate value of the product can be obtained directly from the Table 7B.

In Table 8, the saturated vapour pressure is given as a function of temperature.

In Table 9, the maximum possible duration of bright sunshine hours, N, is given as a function of latitude, for periods of 10 days and for months of the year. Note that the months for the Southern Hemisphere are listed at the bottom of the table.

Equation 32 can now be rewritten with the values obtained from the tables represented by Roman numerals as

$$E = \frac{\frac{I}{59} (0.94 \text{ II} \times \text{III} - \text{IV} \times \text{V} \times \text{VI}) + \text{VII}(\text{VIII} - e_2)}{I + 0.485} \quad (33)$$

Example 1.

As an example, E will now be calculated from the above-mentioned station data. Using the values in the tables we find the following

For T_2	= 23.1 °C	I	= 1.28
For n/N	= 0.65	IIA	= 0.5445
For 42.5 °N in July		III	= 961
For T_2	= 23.1 °C	IV	= 907.0
For e_2	= 12.9 mm Hg	V	= 0.194
For n/N	= 0.65	VI	= 0.72
For u_{10}	= 2.19 m sec ⁻¹	VIIB	= 0.233
For T_2	= 23.1 °C	VIII	= 21.19

When these values are inserted in Eq.33 we get

$$E = \frac{\frac{1.28}{59} (0.94 \times 0.5445 \times 961 - 907 \times 0.194 \times 0.72) + 0.233(21.19 - 12.90)}{1.28 + 0.485}$$

$$= 5.65 \text{ mm day}^{-1} \text{ or } 31 \times 5.65 = 175 \text{ mm during July.}$$

We can also calculate E from the above-mentioned station data, following the scheme set out below and using the tables from Annex I.

	Calculation scheme	Operation	Example
1	T_2	station data	23.1
2	Δ	Table 1	1.28
3	$L = 59$	(59)	59
4	Δ/L	2 : 3	0.022
5	n	station data	9.74
6	N	Table 9	14.96
7	n/N	5 : 6	0.65
8	$0.20 + 0.53 n/N$	Table 2	0.545
9	H_{sh}^{top}	Table 3	961
10	$H_{sh} = (0.20 + 0.53 n/N) H_{sh}^{top}$	8 × 9	524
11	$(1 - \text{albedo}) = 0.94$	(0.94)	0.94
12	$0.94 H_{sh}$	10 × 11	492
13	e_{sat}	Table 8	21.19
14	$e_2 \quad e_2 = h e_{sat}$	station data	12.9
15	$e_{sat} - e_2$	13 - 14	8.29
16	$118.10^{-9} (273 + T_2)^4$	Table 4	907
17	$0.47 - 0.077 \sqrt{e_2}$	Table 5	0.194
18	$0.2 + 0.8 n/N$	Table 6	0.72
19	$H_{10}^n = 118.10^{-9} (273 + T_2)^4 (0.47 - 0.077 \sqrt{e_2}) (0.2 + 0.8 n/N)$	16 × 17 × 18	127
20	$H_o = 0.94 H_{sh} - H_{10}^n$	12 - 19	365
21	$(\Delta/L) H_o$	4 × 20	8.04
22	$\gamma = 0.485$	(0.485)	0.485
23	u_2 or u_{10}	station data	2.19
24	$\gamma f(u_2) = 0.485 \times 0.35(0.5 + 0.54 u_2)$	Table 7	0.233
25	$\gamma E_x = 0.485 \times 0.35(0.5 + 0.54 u_2) (e_{sat} - e_2)$	15 × 24	1.86
26	$(\Delta/L) H_o + \gamma E_x$	21 + 25	9.90
27	$\Delta + \gamma$	2 + 22	1.765
28	$E = \frac{(\Delta/L) H_o + \gamma E_x}{\Delta + \gamma}$ mm/day	26 : 27	5.65

Calculation by means of nomographs

KOOPMANS (1969) has cut Eq.32 into three separate parts

E_1 , the outgoing radiation term, E_2 , the incoming radiation term, and E_3 , the wind term. These terms can be obtained from nomographs.

Nomograph E_1 (Fig.8): Values for T_2 , n/N , and h , the relative humidity, are entered into the nomograph to find the E_1 term; h_2 can be obtained from the temperature (Table 8); E_1 is usually negative.

Nomograph E_2 (Fig.9): Separate nomographs have been prepared for the temperate and tropical climates. The T_2 and n/N values are entered into the nomograph and the obtained point on the empty vertical line in the left-hand side of the figure is connected by a line with the value of H_{sh}^{top} to find the value of E_2 . The necessary value of H_{sh}^{top} can be found in Table 3.

Nomograph E_3 (Fig.10): The values for T_2 and u_2 are connected by a straight line to find the auxiliary point, (as in Nomograph E_2) which then in turn is combined with the value of h to find E_3 . Values of u_{10} or at any other height need to be converted to u_2 values according to Eq.29.

Example 2.

Required data.

$$\left. \begin{array}{l} T_2 = 23.1 \text{ }^\circ\text{C} \rightarrow \text{Table 8} \rightarrow e_{sat} = 21.9 \\ e_2 = 12.9 \end{array} \right\} h = e_2/e_{sat} = 0.61$$

$$n/N = 0.65$$

$$u_2 = u_{10} \frac{\log \frac{100 \times 2}{2}}{\log \frac{100 \times 10}{2}} = 2.19 \times 0.75 = 1.64$$

$$H_{sh}^{top} = 961$$

Lines are drawn on the nomograph for the calculation of the E values. We then find the following values

$$\left. \begin{array}{l} E_1 = -1.5 \\ E_2 = 6.0 \\ E_3 = 1.1 \end{array} \right\} E = E_1 + E_2 + E_3 = -1.5 + 6.0 + 1.1 = 5.6 \text{ mm/day}$$

Nomographs, of course, are less accurate than calculations by means of the tables. However, when calculating machines are not available the nomographs facilitate the calculations.

19.7.2 FORMULA DEVELOPED BY TURC

The original formula by TURC is as follows

$$E_p = \frac{P + 80}{\sqrt{1 + \left(\frac{P+45}{L^{Tc}}\right)^2}} \text{ (mm/10 days)} \quad (34)$$

where

E_p = potential evapotranspiration (mm/10 days)

P = precipitation (mm/10 days)

L^{Tc} = evaporative demand of the atmosphere, calculated according to

$$L^{Tc} = \frac{(T + 2) \sqrt{H_{sh}}}{16} \quad (35)$$

T = average air temperature ($^{\circ}C$) at 2 m

H_{sh} = incoming short-wave radiation ($\text{cal cm}^{-2} \text{ day}^{-1}$)

Calculation by means of nomograms

Nomograms have been developed for the solution of L^{Tc} and E_p (Figs.11 and 12). These nomograms are valid for L^{Tc} values in excess of 10. Smaller values of L^{Tc} cannot be expected in irrigated regions during the growing season. Figure 12 is not suitable for L^{Tc} values greater than 40 or for $P > 80$ mm/10 days. The lines on the nomogram need to be extrapolated for those cases.

Example 3.

We will use the same location as before, with the added information that the average rainfall (P) = 1.8 mm/day, or 18 mm/10 days (note that P and E_p are given per unit of 10 days). The incoming short-wave radiation can be derived from Table 2A and Table 3: for $n/N = 0.65$ we have $H_{sh} = 0.5445 \times 961 = 523 \text{ cal cm}^{-2} \text{ day}^{-1}$. According to Fig.11 for $T = 23.1^{\circ}C$ and $H_{sh} = 523 \text{ cal cm}^{-2} \text{ day}^{-1}$ we obtain $L^{Tc} = 36$. Then we read in Fig.12 for $L^{Tc} = 36$ and $P = 18$ mm/10 days that $E_p = 49$ mm/10 days, or 4.9 mm day^{-1} . Later, TURC simplified Eq.34 to

$$E_p = 0.40 \frac{T}{T+15} (H_{sh} + 50) \text{ (mm/month)} \quad (36)$$

in case the relative humidity, (h), of the air is greater than 50%, and

$$E_p = 0.40 \frac{T}{T+15} (H_{sh} + 50) \left(1 + \frac{50-h}{70}\right) \quad (\text{mm/month}) \quad (37)$$

in case h is less than 50%.

(Note that E_p in Eqs.36 and 37 is given in mm/month.)

For our situation we obtain, according to Eq.36

$$E_p = 0.40 \frac{23.1}{23.1+15} (523+50) = 139 \text{ mm for July, or } 4.49 \text{ mm day}^{-1}.$$

In Sect.7.1 we calculated evaporation rates from open water according to PENMAN of 5.65 mm day^{-1} by means of tables and 5.6 mm day^{-1} by using the nomographs. If we reduce the evaporation from open water surfaces to the potential evapotranspiration by the empirical conversion factor 0.9, we find $0.9 \times 5.65 = 5.08 \text{ mm day}^{-1}$, which is about equal to that calculated according to the original formula by TURC. The newer formula apparently gives a smaller value.

19.7.3 FORMULA DEVELOPED BY JENSEN AND HAISE

The JENSEN-HAISE formula is as follows

$$E_p = (0.025T + 0.08) \frac{H_{sh}}{59} \quad (38)$$

where

E_p = potential evapotranspiration (mm/day)

H_{sh} = incoming short-wave radiation ($\text{cal cm}^{-2} \text{day}^{-1}$)

T = air temperature ($^{\circ}\text{C}$)

Example 4.

Required data

$$T = 23.1 \text{ } ^{\circ}\text{C}$$

$$n/N = 0.65 \rightarrow \text{Tables 2A and 3} \rightarrow H_{sh} = 523 \text{ cal cm}^{-2} \text{day}^{-1}$$

For our example we find that $E_p = (0.025 \times 23.1 + 0.08) \frac{523}{59} = 5.83 \text{ mm.day}^{-1}$.

This is considerably higher than the value for the potential evapotranspiration that was obtained by converting the evaporation from an open water surface with the factor 0.9.

19.7.4 FORMULA DEVELOPED BY BLANEY AND CRIDDLE

According to BLANEY and CRIDDLE

$$E_p = C_1^{BC} p^{BC} (0.457 T + 8.13) \quad (39)$$

where

- E_p = potential evapotranspiration (mm month⁻¹)
- C_1^{BC} = crop coefficient
- T = average air temperature (°C)
- p^{BC} = monthly percentage of daylight hours in the year.

Values of p^{BC} can be obtained from Fig.13. Note that Fig.13 gives the percentage of daylight hours for latitudes north of the equator.

Calculation

The percentage of daylight hours in the Southern Hemisphere can be derived from Fig.13 in the following way. Say one wants to determine the value of p^{BC} for a location with latitude 25 °S for the month of February. To obtain this value, the p^{BC} for August for the same latitude N needs to be multiplied with the ratio of the number of days in the months: in our example, p^{BC} for 25 °N equals 9.08; p^{BC} for February at 25 °S is then equal to 28/31 times 9.08 = 8.20.

Values of C_1^{BC} need to be determined for each crop during its growing season and for every location where one wants to use this formula. For a number of crops the C_1^{BC} coefficients have been determined for the Western United States but these values cannot be applied elsewhere. The determination of the appropriate C_1^{BC} values requires lysimeter data.

Example 5.

In Fig.14 a nomograph is presented for the solution of Eq.38 if the C_1^{BC} coefficients are known. For our example, assuming $C_1^{BC} = 0.85$, and p^{BC} , according to Fig.13, equals 10.4, we have $E_p = 0.85 \times 10.4 (0.475 \times 23.1 + 8.13) = 165$ mm during July, or 5.32 mm day⁻¹. With the aid of Fig.14 a similar value is obtained, which is in this case only slightly higher than the E_p values found with PENMAN's formula.

19.7.5 FORMULA DEVELOPED BY THORNTHWAITE

According to THORNTHWAITE

$$E_p = 1.6 \left(\frac{10T}{I} \right)^a \quad (40)$$

where

- E_p = potential evapotranspiration (cm month⁻¹, in months of 30 days, with a day-length of 12 hours)
 T = average air temperature (°C), calculated from daily means
 I = annual heat index, i.e. the sum of the 12 monthly heat indices, i , where

$$i = \left(\frac{T}{5} \right)^{1.514} \quad (41)$$

$$a = 0.000000675 I^3 - 0.000077 I^2 + 0.01792 I + 0.49239 \quad (42)$$

Calculation

The monthly heat index i can be obtained for a given temperature from Table 10. Summation of the monthly heat indices gives the value of I . This value of I is entered into Fig.15 on the I scale, and a straight line is drawn through I and the point of convergence. The mean monthly temperature is then used as argument for entering the nomograph. For each temperature the corresponding value of the potential evapotranspiration is found on the x-axis. This is the potential evapotranspiration for a 30-day month with day-lengths of 12 hours. The correction factor to be used to convert this, depending on the latitude and month, can be found in Table 11. Figure 15 cannot be used for temperatures in excess of 26.5 °C. The uncorrected value of the potential evapotranspiration for these higher temperatures can be found in Table 12.

Example 6.

We will calculate the potential evapotranspiration according to THORNTHWAITE's method again for our example at 42.5 °N during the month of July. We need to know the mean air temperature also for the other months in order to calculate the annual heat index I .

	T	i
January	- 4.8	0
February	1.7	0.20
March	5.5	1.16
April	12.0	3.76
May	16.6	6.15
June	21.3	8.97
<u>July</u>	<u>23.1</u>	10.15
August	24.2	10.89
September	19.2	7.67
October	11.9	3.72
November	11.2	3.39
December	1.6	<u>0.18</u>
Annual heat index I = 56.24		

The straight line in Fig.15 is drawn through the value of 56.24 for I and the point of convergence. For a temperature of 23.1 °C we find then an uncorrected value $E_p = 115$ mm. The correction factor found in Table 11 for 42.5 °N in July equals 1.28. Hence the potential evapotranspiration $E_p = 1.28 \times 115 = 147$ mm or 4.74 mm day^{-1} , which in this case is only slightly lower than the value calculated with the aid of PENMAN's formula. Generally, THORNTHWAITE's formula gives values for the evapotranspiration which are too low. This is also evident from Figs. 16 and 17, in which measured pan evaporation (pans with different exposures) is compared with the evapotranspiration from a lysimeter (with short grass cover) and with calculated evaporation values, for Cagua (Venezuela), 10 °S, during 1963. Lysimeter data were available for only one part of the year. During the hottest months the potential evapotranspiration values based on TURC's and JENSEN and HAISE's formulas fall considerably short of the open pan evaporation. Even PENMAN's estimate is too low during that period. The formulas by BLANEY and CRIDDLE and by THORNTHWAITE are clearly not suitable for the monsoon-type climate of Northern Venezuela.

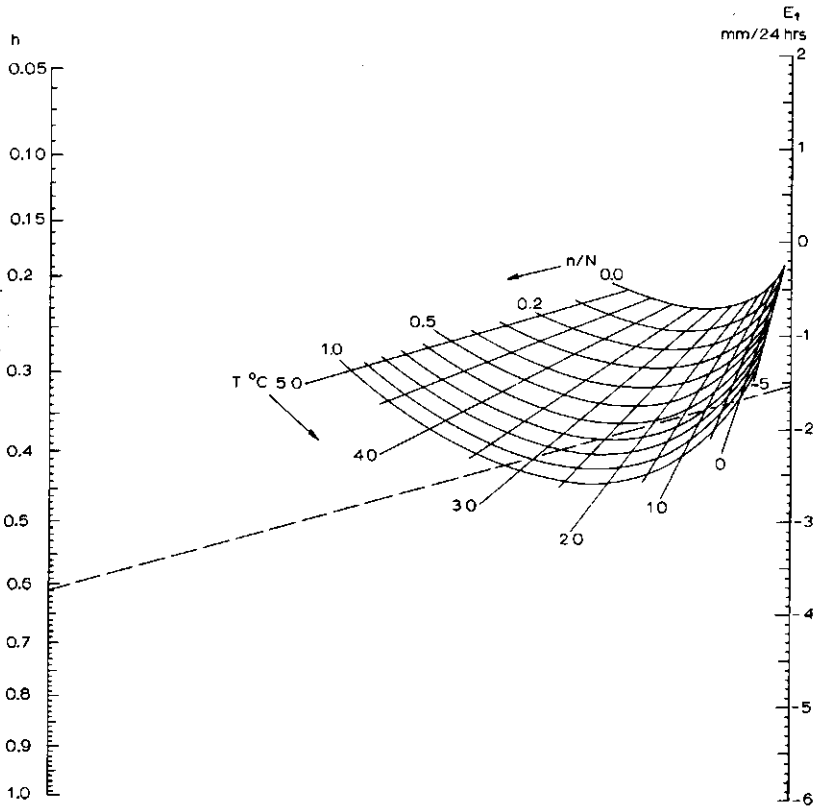
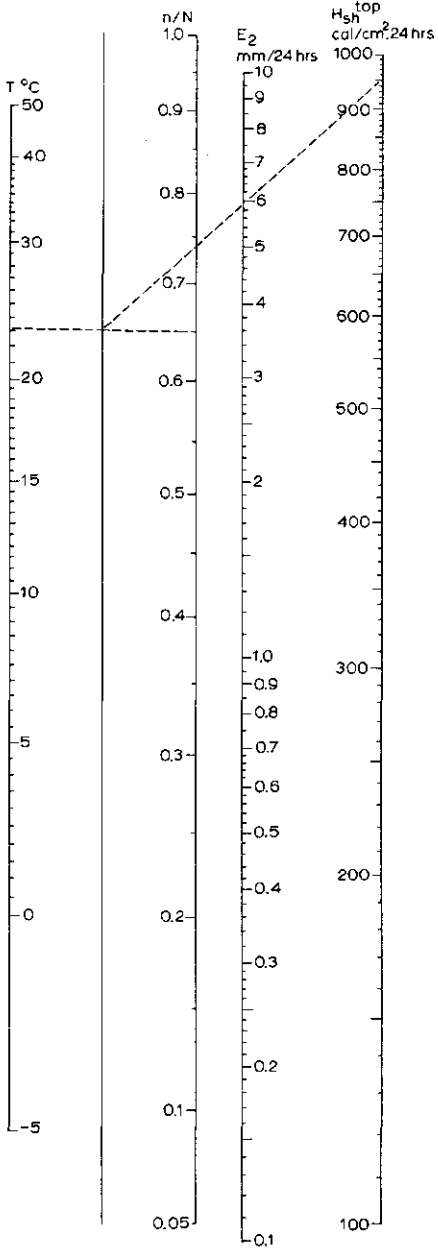


Fig.8. Outgoing radiation, E_1

TEMPERATE CLIMATES



TROPICAL CLIMATES

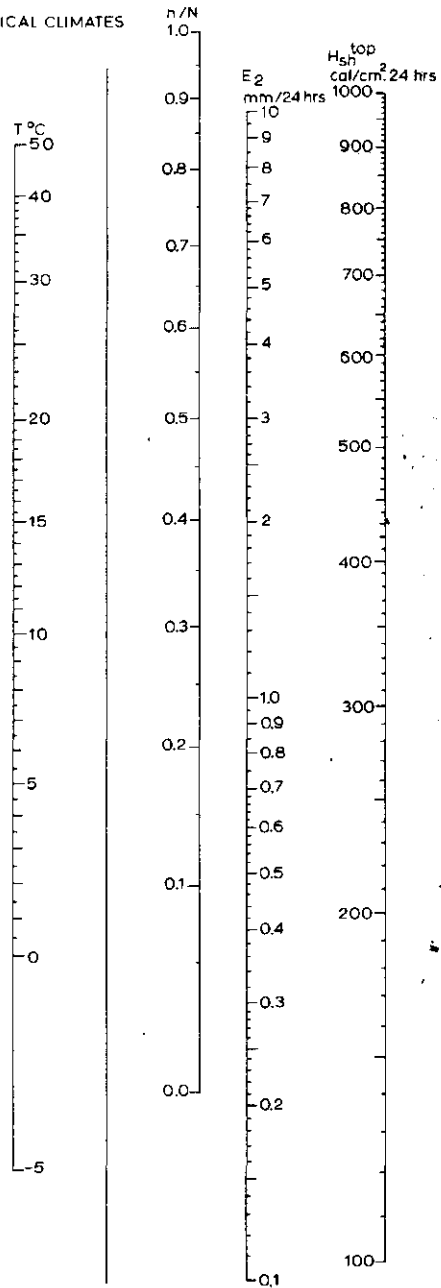


Fig.9. Incoming radiation, E_2 , for open water

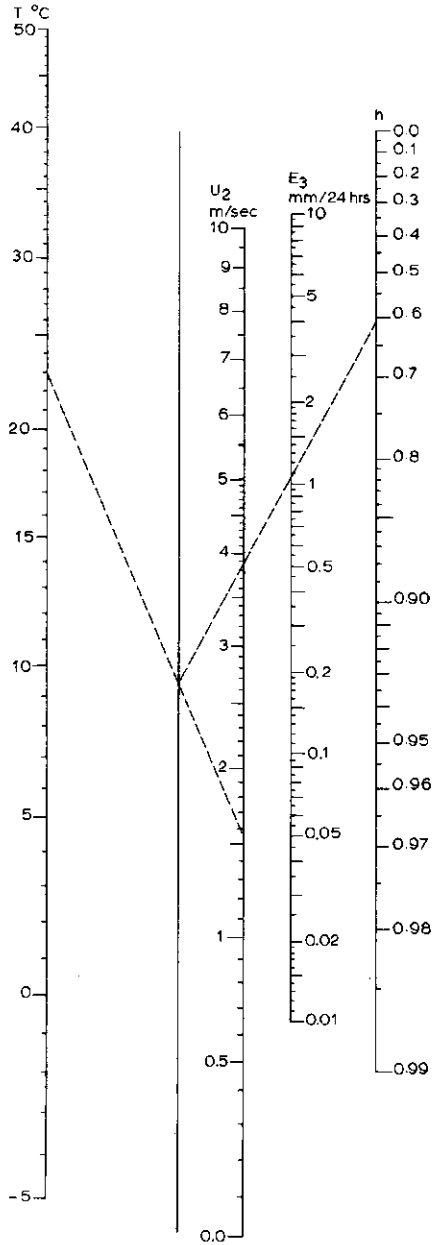


Fig.10. Windterm E_3

evaporation capacity
of the air

L^{Tc}

52

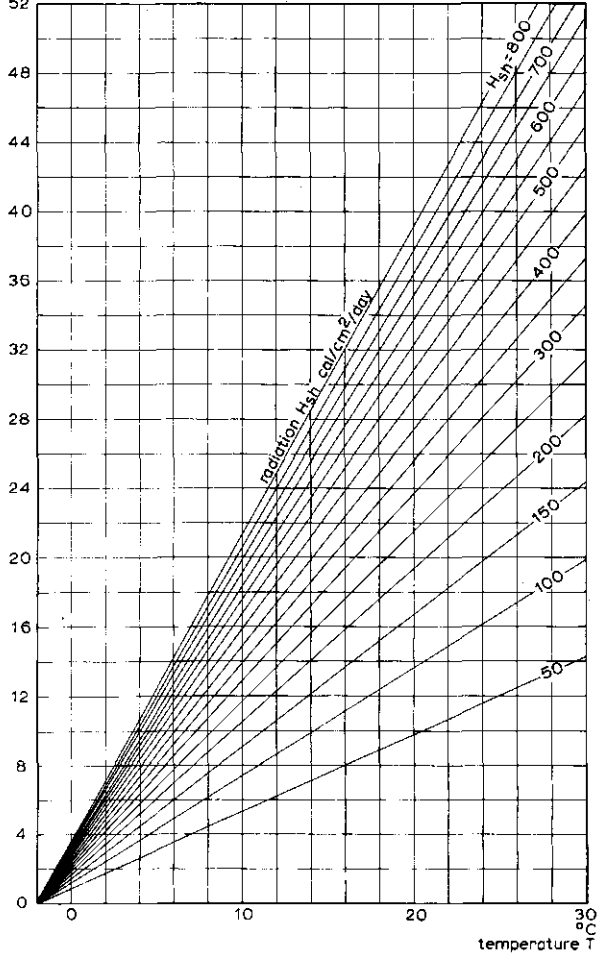


Fig.11. Nomogram for determining the evaporation capacity of the air L^{Tc} (VAN BEERS)

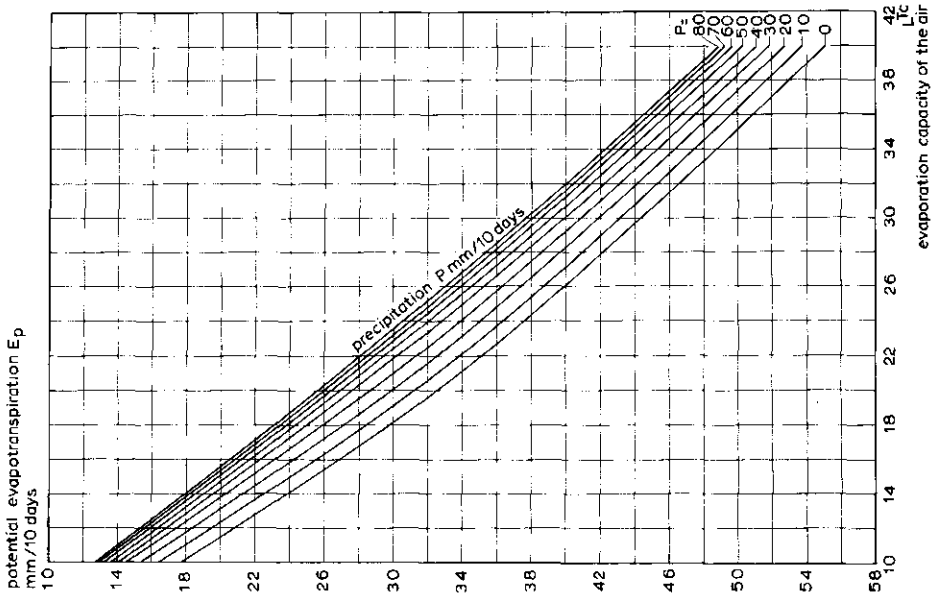


Fig.12. Nomogram for determining the potential evapotranspiration E_p (VAN BEERS)

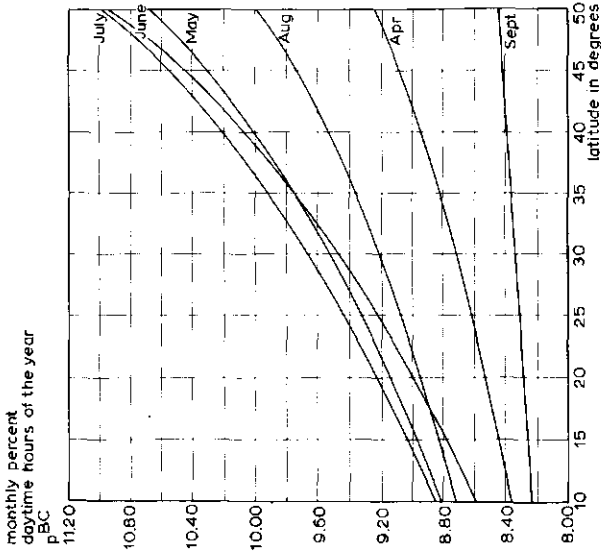


Fig.13. Monthly percent of daytime hours of the year for latitudes 10 to 50 degrees north of the equator (BLANEY and CRIDDLE, 1950)

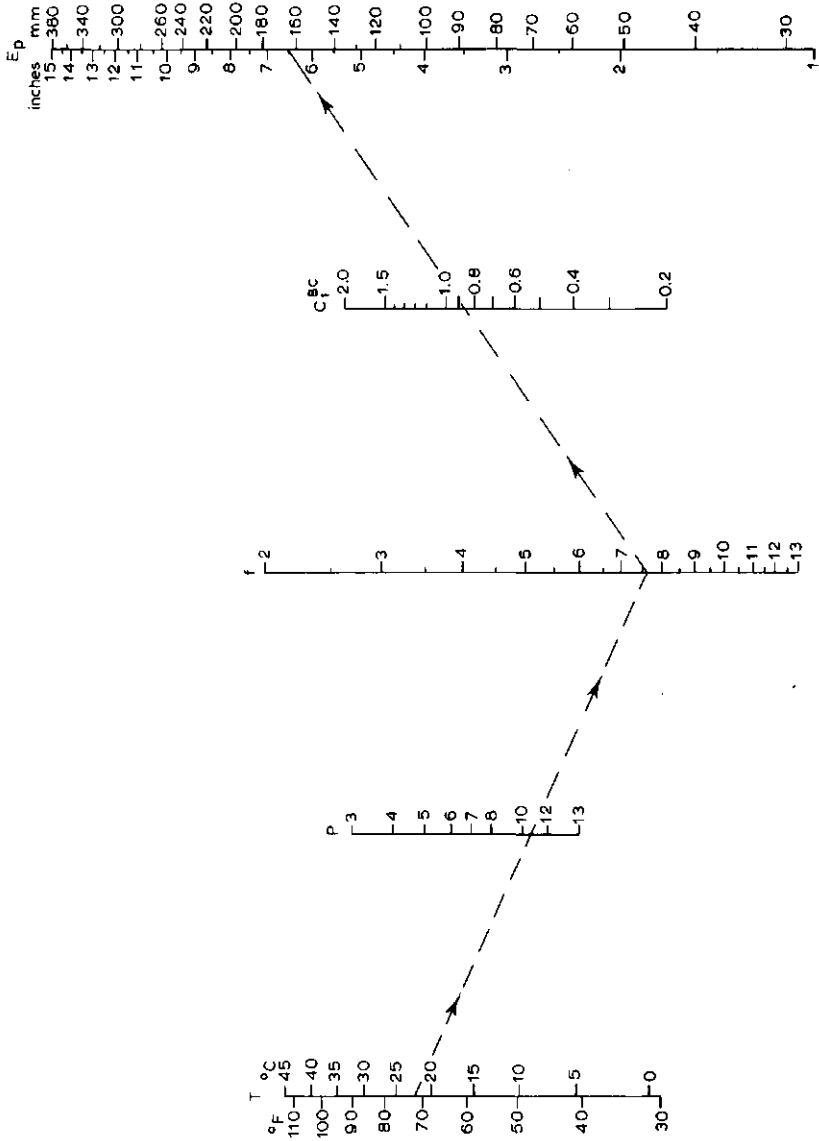


Fig.14. Nomograph for determining the monthly evapotranspiration E_p (BLANEY and CRIDDLE, 1962)

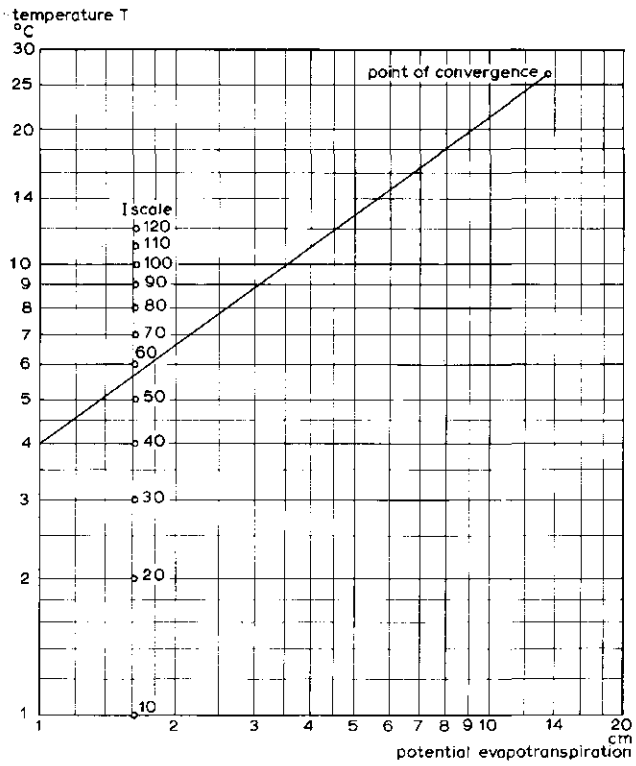


Fig.15. Nomograph for determining potential evapotranspiration (THORNTHWAITE, 1948)

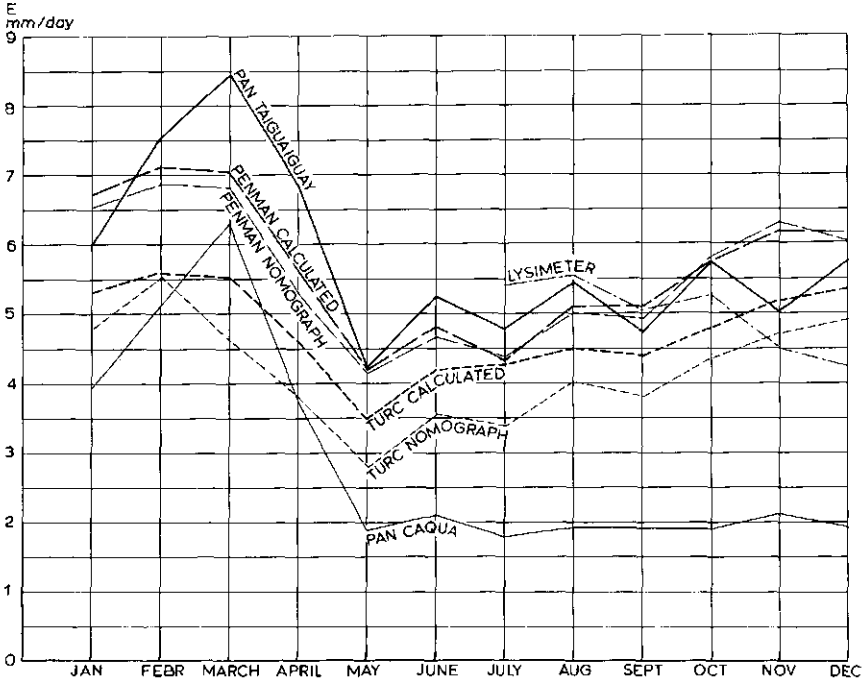


Fig.16. Calculated and measured (pot.) evaporation (Venezuela 1963)

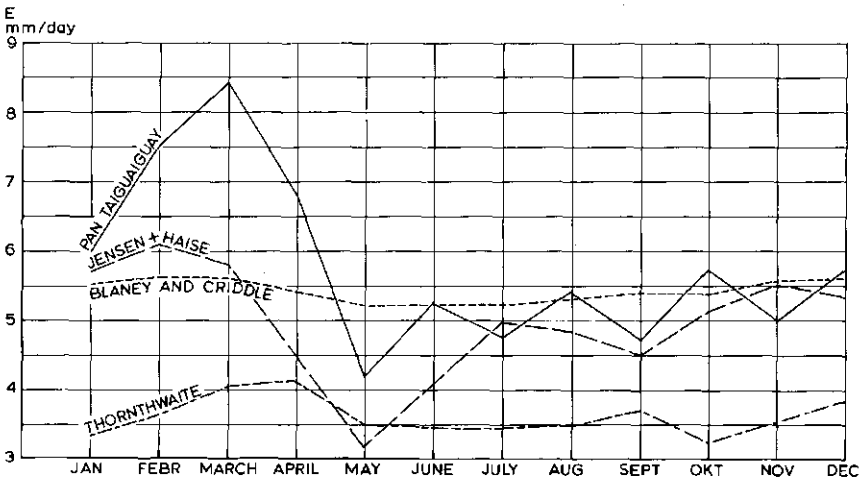


Fig.17. Calculated and measured (pot.) evaporation (Venezuela 1963)

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ANNEX I.
TABLES FOR CALCULATING EVAPORATION AND EVAPOTRANSPIRATION

Table 1. Values of Δ (change of saturated vapour pressure with temperature, mm Hg °C⁻¹), based on data from Handbook of Chemistry and Physics, 49th Ed., pages D-109.

T ₂	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	T ₂
-10	0,17	0,17	0,17	0,17	0,17	0,16	0,16	0,16	0,16	0,16	-10
-9	0,18	0,18	0,18	0,18	0,18	0,18	0,18	0,17	0,17	0,17	-9
-8	0,20	0,19	0,19	0,19	0,19	0,19	0,19	0,19	0,19	0,18	-8
-7	0,21	0,21	0,21	0,21	0,20	0,20	0,20	0,20	0,20	0,20	-7
-6	0,22	0,22	0,22	0,22	0,22	0,22	0,22	0,21	0,21	0,21	-6
-5	0,24	0,24	0,24	0,24	0,23	0,23	0,23	0,23	0,23	0,23	-5
-4	0,26	0,26	0,25	0,25	0,25	0,25	0,25	0,25	0,24	0,24	-4
-3	0,27	0,27	0,27	0,27	0,27	0,27	0,26	0,26	0,26	0,26	-3
-2	0,29	0,29	0,29	0,29	0,28	0,28	0,28	0,28	0,28	0,28	-2
-1	0,31	0,31	0,31	0,31	0,30	0,30	0,30	0,30	0,30	0,29	-1
0	0,33	0,33	0,33	0,33	0,32	0,32	0,32	0,32	0,32	0,31	0
0	0,34	0,34	0,34	0,34	0,34	0,35	0,35	0,35	0,35	0,35	0
1	0,36	0,36	0,36	0,36	0,36	0,37	0,37	0,37	0,37	0,38	1
2	0,38	0,38	0,38	0,39	0,39	0,39	0,39	0,40	0,40	0,40	2
3	0,40	0,41	0,41	0,41	0,41	0,42	0,42	0,42	0,42	0,43	3
4	0,43	0,43	0,43	0,44	0,44	0,44	0,44	0,45	0,45	0,45	4
5	0,45	0,46	0,46	0,46	0,46	0,47	0,47	0,47	0,48	0,48	5
6	0,48	0,49	0,49	0,49	0,50	0,50	0,50	0,51	0,51	0,51	6
7	0,52	0,52	0,52	0,53	0,53	0,53	0,54	0,54	0,54	0,55	7
8	0,55	0,55	0,56	0,56	0,56	0,57	0,57	0,57	0,58	0,58	8
9	0,58	0,59	0,59	0,59	0,60	0,60	0,60	0,61	0,61	0,62	9
10	0,62	0,62	0,63	0,63	0,64	0,64	0,64	0,65	0,65	0,65	10
11	0,66	0,66	0,66	0,67	0,67	0,68	0,68	0,68	0,69	0,69	11
12	0,69	0,70	0,70	0,71	0,71	0,71	0,72	0,72	0,73	0,73	12
13	0,73	0,74	0,74	0,75	0,75	0,76	0,76	0,77	0,77	0,77	13
14	0,78	0,78	0,79	0,79	0,79	0,80	0,80	0,81	0,81	0,82	14
15	0,82	0,83	0,83	0,84	0,84	0,85	0,85	0,86	0,86	0,87	15
16	0,87	0,88	0,88	0,89	0,89	0,90	0,90	0,91	0,91	0,92	16
17	0,92	0,93	0,93	0,94	0,94	0,95	0,95	0,96	0,96	0,97	17
18	0,97	0,98	0,98	0,99	0,99	1,00	1,00	1,01	1,01	1,02	18
19	1,03	1,04	1,04	1,05	1,05	1,06	1,06	1,07	1,07	1,08	19
20	1,08	1,09	1,09	1,10	1,10	1,11	1,12	1,12	1,13	1,14	20
21	1,14	1,15	1,16	1,16	1,17	1,18	1,18	1,19	1,20	1,20	21
22	1,21	1,22	1,22	1,23	1,24	1,24	1,25	1,26	1,26	1,27	22
23	1,28	1,28	1,29	1,30	1,30	1,31	1,32	1,32	1,33	1,34	23
24	1,34	1,35	1,36	1,36	1,37	1,38	1,38	1,39	1,40	1,40	24
25	1,41	1,42	1,42	1,43	1,44	1,45	1,46	1,47	1,48	1,49	25
26	1,49	1,50	1,51	1,52	1,53	1,53	1,54	1,55	1,56	1,57	26
27	1,57	1,58	1,59	1,60	1,60	1,61	1,62	1,63	1,63	1,64	27
28	1,65	1,66	1,67	1,68	1,69	1,70	1,71	1,72	1,72	1,73	28
29	1,74	1,75	1,76	1,77	1,78	1,79	1,79	1,80	1,81	1,81	29
30	1,82	1,83	1,84	1,85	1,86	1,87	1,88	1,89	1,90	1,91	30
31	1,92	1,93	1,94	1,95	1,96	1,97	1,98	1,99	2,00	2,01	31
32	2,02	2,03	2,04	2,05	2,06	2,07	2,08	2,09	2,10	2,11	32
33	2,12	2,13	2,14	2,15	2,16	2,17	2,18	2,19	2,20	2,21	33
34	2,22	2,23	2,24	2,26	2,27	2,28	2,29	2,30	2,31	2,33	34
35	2,34	2,35	2,36	2,37	2,38	2,39	2,40	2,41	2,42	2,44	35
36	2,45	2,46	2,47	2,48	2,49	2,51	2,52	2,53	2,54	2,55	36
37	2,57	2,58	2,59	2,60	2,61	2,63	2,64	2,65	2,66	2,68	37
38	2,69	2,70	2,71	2,72	2,73	2,75	2,76	2,78	2,79	2,81	38
39	2,82	2,83	2,84	2,85	2,86	2,88	2,89	2,90	2,91	2,93	39
40	2,94	2,95	2,97	2,98	2,99	3,00	3,01	3,02	3,04	3,05	40

Read for the comma a decimal point

Table 2A. Values of $0.20 + 0.53 \frac{n}{N}$ (temperate climates)

$\frac{n}{N}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	$\frac{n}{N}$
0,00	0,2000	0,2053	0,2106	0,2159	0,2212	0,2265	0,2313	0,2371	0,2424	0,2477	0,00
0,10	0,2530	0,2583	0,2636	0,2689	0,2742	0,2795	0,2848	0,2901	0,2954	0,3007	0,10
0,20	0,3060	0,3113	0,3166	0,3219	0,3272	0,3325	0,3378	0,3431	0,3484	0,3537	0,20
0,30	0,3590	0,3643	0,3696	0,3749	0,3802	0,3855	0,3908	0,3961	0,4014	0,4067	0,30
0,40	0,4120	0,4173	0,4226	0,4279	0,4332	0,4385	0,4438	0,4491	0,4544	0,4597	0,40
0,50	0,4650	0,4703	0,4756	0,4809	0,4862	0,4915	0,4968	0,5021	0,5074	0,5127	0,50
0,60	0,5180	0,5233	0,5286	0,5339	0,5392	0,5445	0,5498	0,5551	0,5604	0,5657	0,60
0,70	0,5710	0,5763	0,5816	0,5869	0,5922	0,5975	0,6023	0,6081	0,6134	0,6187	0,70
0,80	0,6240	0,6293	0,6346	0,6399	0,6452	0,6505	0,6558	0,6611	0,6664	0,6717	0,80
0,90	0,6770	0,6823	0,6876	0,6929	0,6982	0,7035	0,7088	0,7141	0,7194	0,7247	0,90
1,00	0,7300										
$\frac{n}{N}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	$\frac{n}{N}$

Read for the comma a decimal point

Table 2B. Values of $0.28 + 0.48 \frac{n}{N}$ (subtropical and tropical climates)

$\frac{n}{N}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	$\frac{n}{N}$
0,00	0,2800	0,2848	0,2896	0,2944	0,2992	0,3040	0,3088	0,3136	0,3184	0,3232	0,00
0,10	0,3280	0,3328	0,3376	0,3424	0,3472	0,3520	0,3568	0,3616	0,3664	0,3712	0,10
0,20	0,3760	0,3808	0,3856	0,3904	0,3952	0,4000	0,4048	0,4096	0,4144	0,4192	0,20
0,30	0,4240	0,4288	0,4336	0,4384	0,4432	0,4480	0,4528	0,4576	0,4624	0,4672	0,30
0,40	0,4720	0,4768	0,4816	0,4864	0,4912	0,4960	0,5008	0,5056	0,5104	0,5152	0,40
0,50	0,5200	0,5248	0,5296	0,5344	0,5392	0,5440	0,5488	0,5536	0,5584	0,5632	0,50
0,60	0,5680	0,5728	0,5776	0,5824	0,5872	0,5920	0,5968	0,6016	0,6064	0,6112	0,60
0,70	0,6160	0,6208	0,6256	0,6304	0,6352	0,6400	0,6448	0,6496	0,6544	0,6592	0,70
0,80	0,6640	0,6688	0,6736	0,6784	0,6832	0,6880	0,6928	0,6976	0,7024	0,7072	0,80
0,90	0,7120	0,7168	0,7216	0,7264	0,7312	0,7360	0,7408	0,7456	0,7504	0,7552	0,90
1,00	0,7600										1,00
$\frac{n}{N}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	$\frac{n}{N}$

Read for the comma a decimal point

Table 4. Values of $118.10^{-9} (273 + T_2)^4$

T ₂	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	T ₂
-10	564,6										-10
-9	573,2	572,3	571,5	570,6	569,8	568,9	568,0	567,2	566,3	565,5	-9
-8	581,9	581,0	580,2	579,3	578,4	577,6	576,7	575,8	574,9	574,1	-8
-7	590,8	589,9	589,0	588,1	587,2	586,4	585,5	584,6	583,7	582,8	-7
-6	599,7	598,8	597,9	597,0	596,1	595,3	594,4	593,5	592,6	591,7	-6
-5	608,7	607,8	606,9	606,0	605,1	604,2	603,3	602,4	601,5	600,6	-5
-4	618,9	617,9	616,9	615,8	614,8	613,8	612,8	611,8	610,7	609,7	-4
-3	627,1	626,3	625,5	624,6	623,8	623,0	622,2	621,4	620,5	619,7	-3
-2	636,4	635,5	634,5	633,6	632,7	631,8	630,8	629,9	629,0	628,0	-2
-1	645,9	645,0	644,0	643,1	642,1	641,2	640,2	639,3	638,3	637,4	-1
0	655,4	654,4	653,2	652,5	651,6	650,6	649,7	648,8	647,8	646,9	0
0	655,4	656,4	657,3	658,3	659,3	660,2	661,2	662,2	663,2	664,1	0
1	665,1	666,1	667,1	668,1	669,0	670,0	671,0	672,0	673,0	674,0	1
2	674,9	675,9	676,9	677,9	678,8	679,8	680,8	681,8	682,8	683,7	2
3	684,7	685,7	686,7	687,7	688,7	689,7	690,7	691,7	692,7	693,7	3
4	694,7	695,7	696,7	697,7	698,7	699,7	700,0	701,8	702,8	703,8	4
5	704,8	705,8	706,8	707,8	708,8	709,9	710,9	711,9	712,9	714,0	5
6	715,0	716,0	717,1	718,1	719,1	720,2	721,2	722,2	723,3	724,3	6
7	725,3	726,3	727,4	728,4	729,5	730,5	731,5	732,6	733,6	734,7	7
8	735,7	736,8	737,8	738,9	739,4	741,0	742,0	743,1	744,1	745,2	8
9	746,2	747,3	748,3	749,4	750,5	751,6	752,7	753,8	754,9	755,9	9
10	756,9	758,0	759,0	760,1	761,2	762,3	763,3	764,4	765,5	766,5	10
11	767,7	768,7	769,8	770,9	772,0	773,0	774,1	775,2	776,3	777,4	11
12	778,5	779,6	780,7	781,8	782,9	784,0	785,1	786,2	787,3	788,4	12
13	789,5	790,6	791,7	792,8	793,9	795,0	796,1	797,2	798,3	799,4	13
14	800,5	801,6	802,8	803,9	805,0	806,2	807,3	808,4	809,5	810,7	14
15	811,8	812,9	814,1	815,2	816,3	817,5	818,6	819,7	820,8	822,0	15
16	823,1	824,3	825,4	826,6	827,7	828,9	830,0	831,2	832,3	833,5	16
17	834,6	835,8	836,9	838,1	839,2	840,4	841,8	842,7	843,9	845,0	17
18	846,2	847,4	848,5	849,7	850,9	852,1	853,2	854,4	855,6	856,7	18
19	857,9	859,1	860,3	861,4	862,6	863,8	865,0	866,2	867,3	868,5	19
20	869,7	870,9	872,1	873,3	874,5	875,7	876,8	878,0	879,2	880,4	20
21	881,6	882,8	884,0	885,2	886,4	887,8	888,9	890,1	891,8	892,5	21
22	893,7	894,9	896,1	897,3	898,5	899,8	901,0	902,2	903,4	904,5	22
23	905,8	907,0	908,3	909,5	910,7	912,0	913,2	914,4	915,6	916,9	23
24	918,1	919,4	920,6	921,9	923,1	924,4	925,6	926,9	928,1	929,4	24
25	930,6	931,9	933,1	934,4	935,6	936,9	938,1	939,4	940,6	941,9	25
26	943,1	944,4	945,6	946,9	948,2	949,5	950,7	952,0	953,3	954,5	26
27	955,8	957,1	958,4	959,6	960,9	962,2	963,5	964,8	966,0	967,3	27
28	968,6	969,9	971,2	972,5	973,8	975,1	976,3	977,6	978,9	980,2	28
29	981,5	982,8	984,1	985,4	986,7	988,1	989,4	990,7	992,0	993,3	29
30	994,6	995,9	997,2	998,5	999,8	1001,2	1002,5	1003,8	1005,1	1006,5	30
31	1007,8	1009,1	1010,5	1011,8	1013,1	1014,4	1015,8	1016,1	1018,5	1019,8	31
32	1021,1	1022,4	1023,8	1025,1	1026,5	1027,8	1029,2	1030,5	1031,9	1033,2	32
33	1034,6	1035,9	1037,3	1038,6	1040,0	1041,3	1042,7	1044,0	1045,4	1046,7	33
34	1048,2	1049,5	1050,9	1052,2	1053,6	1054,9	1056,4	1057,7	1059,1	1060,4	34
35	1061,9	1063,3	1064,7	1066,0	1067,4	1068,7	1070,2	1071,6	1073,0	1074,4	35
36	1075,8	1077,2	1078,6	1080,0	1081,3	1082,7	1084,1	1085,5	1086,9	1088,4	36
37	1089,8	1091,2	1092,6	1094,0	1095,4	1096,8	1098,2	1099,6	1101,0	1102,4	37
38	1103,9	1105,3	1106,7	1108,1	1109,5	1110,9	1112,4	1113,8	1115,3	1116,7	38
39	1118,2	1119,6	1121,0	1122,4	1123,9	1125,3	1126,8	1128,2	1129,7	1131,1	39
40	1132,6	1134,0	1135,5	1136,9	1138,4	1139,8	1141,3	1142,7	1144,2	1145,6	40
T ₂	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	T ₂

Read for the comma a decimal point

Table 3. Incoming extra-terrestrial radiation, H_{sh}^{top} , ($\text{cal cm}^{-2} \text{ day}^{-1}$)
 (Smithsonian Meteorological Tables, table 132)

ϕ_N	month 10 d.	Jan.			Febr.			March			April			May			June		
		I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III
80	10 d. month							45	105	195	310	455	615	750	865	960	1020	1050	1060
	10 d. month								115						860		1045		
70	10 d. month				30	75	125	185	260	345	445	550	650	740	840	915	975	1010	1010
	10 d. month					75			265						830		1000		
60	10 d. month	60	80	110	150	210	270	335	405	480	565	655	735	805	865	920	955	980	975
	10 d. month		85			210			405						865		970		
50	10 d. month	190	215	250	300	355	415	475	540	605	680	745	805	860	910	945	970	985	985
	10 d. month		215			355			540						905		980		
40	10 d. month	335	360	395	440	495	550	600	655	710	765	820	865	905	940	965	980	990	990
	10 d. month		365			495			655						935		985		
30	10 d. month	480	500	535	575	620	665	710	750	795	835	875	905	930	950	965	970	975	975
	10 d. month		505			620			750						950		975		
20	10 d. month	615	635	665	695	730	765	795	825	850	875	895	910	920	930	935	935	935	935
	10 d. month		640			730			825						930		935		
10	10 d. month	745	760	775	800	820	840	860	875	890	895	900	895	895	890	885	880	875	870
	10 d. month		760			820			875						890		875		
0	10 d. month	850	860	870	880	890	895	900	895	890	880	870	860	840	825	810	800	795	790
	10 d. month		860			890			895						825		795		
10	10 d. month	935	935	935	935	930	925	910	895	870	845	820	790	760	735	715	700	690	690
	10 d. month		935			930			895						730		695		
20	10 d. month	995	990	980	965	945	920	890	860	825	785	745	700	665	630	605	580	570	570
	10 d. month		990			945			860						635		575		
30	10 d. month	1035	1025	1000	970	930	890	850	805	750	700	645	595	550	510	480	455	440	440
	10 d. month		1020			930			800						515		445		
40	10 d. month	1050	1025	990	950	895	835	780	720	660	590	530	470	420	380	340	315	305	300
	10 d. month		1020			895			720						380		305		
50	10 d. month	1040	1010	965	905	835	765	690	620	545	465	395	335	285	240	205	180	170	165
	10 d. month		1005			835			620						245		170		
60	10 d. month	1020	980	920	840	755	665	580	495	415	330	255	195	145	110	80	60	50	50
	10 d. month		975			755			495						110		55		
70	10 d. month	1040	985	900	780	660	550	455	360	270	190	115	55	25					
	10 d. month		975			665			360						10				
80	10 d. month	1090	1030	925	785	620	460	325	210	125	50								
	10 d. month		1015			620			220										
ϕ_S	10 d. month	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III
		Jan.			Febr.			March			April			May			June		

Evapotranspiration

(Table 3 cont.)

	July			August			Sept.			Oct.			Nov.			Dec.		
	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III
1035	980	900	795	670	505	355	230	135	65	10								
	970		655				240			25								
980	935	860	770	670	570	470	375	290	210	135	80	35						
	925		670				380			140			10					
960	930	885	825	750	670	590	510	430	350	280	215	160	110	85	60	50	50	
	925		750				510			280			120			55		
975	955	920	875	820	760	695	625	555	490	420	360	305	255	220	195	180	180	
	950		820				625			425			260			185		
980	970	940	910	870	825	775	720	665	605	550	495	445	400	360	335	320	320	
	965		870				720			550			400			325		
970	960	950	930	905	875	840	800	755	710	670	620	580	540	510	485	470	470	
	960		905				800			665			545			475		
935	930	925	920	910	895	875	855	830	795	760	725	695	665	640	625	610	610	
	930		910				855			760			665			615		
875	880	880	885	890	890	890	880	870	855	840	820	800	775	760	745	735	735	
	880		890				880			840			780			740		
795	800	815	830	844	855	870	880	885	890	890	885	870	860	855	850	845	845	
	805		840				880			890			860			845		
695	710	725	745	770	800	830	850	875	895	910	920	925	930	930	930	930	930	
	710		770				850			910			930			930		
575	590	615	645	680	720	760	800	840	870	905	930	950	970	980	990	1000	1000	
	595		680				800			900			965			995		
450	470	490	530	570	620	675	725	775	825	870	915	955	985	1010	1030	1040	1040	
	470		575				725			870			985			1035		
305	325	360	400	450	505	565	625	690	750	810	870	925	970	1010	1040	1055	1060	
	330		450				625			810			970			1050		
175	190	220	260	310	370	440	510	585	660	740	810	885	945	995	1030	1050	1055	
	195		315				510			735			940			1045		
55	70	95	130	175	230	300	375	460	550	640	730	820	895	960	1010	1040	1045	
	75		180				380			640			890			1030		
			50	95	160	230	325	420	525	635	745	860	950	1020	1070	1080		
			50				240			525			850			1055		
						35	95	175	280	410	565	740	895	1000	1070	1115	1130	
							100			420			880			1105		
I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	
	July		August			Sept.			Oct.			Nov.			Dec.			

Table 5. Values of $0.47 - 0.077 \sqrt{e_2}$

e_2	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	e_2
0	0,470	0,445	0,435	0,428	0,421	0,415	0,411	0,405	0,401	0,397	0
1	0,393	0,389	0,385	0,382	0,379	0,376	0,373	0,370	0,367	0,364	1
2	0,361	0,358	0,356	0,353	0,351	0,348	0,346	0,344	0,341	0,339	2
3	0,337	0,334	0,332	0,330	0,328	0,326	0,324	0,322	0,320	0,318	3
4	0,316	0,314	0,312	0,311	0,308	0,307	0,305	0,303	0,301	0,300	4
5	0,298	0,296	0,294	0,293	0,291	0,289	0,288	0,286	0,284	0,283	5
6	0,281	0,280	0,278	0,277	0,275	0,274	0,272	0,271	0,269	0,267	6
7	0,266	0,265	0,264	0,262	0,261	0,259	0,257	0,257	0,255	0,254	7
8	0,252	0,251	0,250	0,248	0,247	0,245	0,244	0,243	0,241	0,241	8
9	0,239	0,237	0,237	0,235	0,234	0,233	0,231	0,231	0,229	0,227	9
10	0,227	0,225	0,224	0,223	0,221	0,221	0,219	0,218	0,217	0,216	10
11	0,214	0,214	0,212	0,211	0,210	0,209	0,207	0,207	0,205	0,204	11
12	0,204	0,202	0,201	0,200	0,199	0,197	0,197	0,196	0,194	0,194	12
13	0,192	0,191	0,190	0,189	0,188	0,187	0,186	0,185	0,184	0,183	13
14	0,182	0,180	0,180	0,179	0,178	0,177	0,176	0,175	0,174	0,173	14
15	0,172	0,170	0,170	0,169	0,168	0,167	0,166	0,165	0,164	0,163	15
16	0,162	0,161	0,160	0,159	0,158	0,157	0,157	0,155	0,154	0,154	16
17	0,153	0,151	0,150	0,150	0,149	0,148	0,147	0,146	0,145	0,144	17
18	0,144	0,143	0,141	0,140	0,140	0,139	0,138	0,137	0,136	0,135	18
19	0,134	0,134	0,133	0,132	0,131	0,130	0,129	0,128	0,127	0,127	19
20	0,126	0,125	0,124	0,123	0,122	0,121	0,120	0,120	0,119	0,118	20
21	0,117	0,117	0,116	0,114	0,113	0,113	0,112	0,111	0,110	0,110	21
22	0,109	0,108	0,107	0,107	0,106	0,105	0,104	0,103	0,103	0,101	22
23	0,100	0,100	0,099	0,098	0,097	0,097	0,096	0,095	0,094	0,093	23
24	0,093	0,092	0,091	0,090	0,090	0,089	0,088	0,087	0,087	0,086	24
25	0,085	0,084	0,083	0,083	0,082	0,081	0,080	0,080	0,079	0,078	25
26	0,077	0,077	0,076	0,075	0,074	0,073	0,073	0,072	0,071	0,070	26
27	0,070	0,069	0,068	0,068	0,067	0,067	0,066	0,065	0,064	0,063	27
28	0,063	0,062	0,061	0,060	0,060	0,059	0,058	0,057	0,057	0,056	28
29	0,055	0,055	0,054	0,053	0,053	0,052	0,051	0,050	0,050	0,049	29
30	0,048	0,047	0,047	0,047	0,046	0,045	0,044	0,043	0,043	0,042	30
e_2	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	e_2

Read for the comma a decimal point

Table 6. Values of $0.2 + 0.8 \frac{n}{N}$

Evapotranspiration

$\frac{n}{N}$,00	,01	,02	,03	,04	,05	,06	,07	,08	,09	$\frac{n}{N}$
0,00	0,20	0,21	0,22	0,22	0,23	0,24	0,25	0,26	0,26	0,27	0,00
0,10	0,28	0,29	0,30	0,30	0,31	0,32	0,33	0,34	0,34	0,35	0,10
0,20	0,36	0,37	0,38	0,38	0,39	0,40	0,41	0,42	0,42	0,43	0,20
0,30	0,44	0,45	0,46	0,46	0,47	0,48	0,49	0,50	0,50	0,51	0,30
0,40	0,52	0,53	0,54	0,54	0,55	0,56	0,57	0,58	0,58	0,59	0,40
0,50	0,60	0,61	0,62	0,62	0,63	0,64	0,65	0,66	0,66	0,67	0,50
0,60	0,68	0,69	0,70	0,70	0,71	0,72	0,73	0,74	0,74	0,75	0,60
0,70	0,76	0,77	0,78	0,78	0,79	0,80	0,81	0,82	0,82	0,83	0,70
0,80	0,84	0,85	0,86	0,86	0,87	0,88	0,89	0,90	0,90	0,91	0,80
0,90	0,92	0,93	0,94	0,94	0,95	0,96	0,97	0,98	0,98	0,99	0,90
1,00	1,00										1,00

Read for the comma a decimal point

Table 7A. Values of $0.485 \times 0.35 (0.5 + 0.54 u)$ for u ($m \text{ sec}^{-1}$) measured at 2 m

u_{200} m/sec	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	u_{200} m/sec
0	0,085	0,094	0,103	0,112	0,121	0,131	0,140	0,149	0,158	0,167	0
1	0,177	0,186	0,195	0,204	0,213	0,222	0,231	0,240	0,249	0,259	1
2	0,268	0,277	0,286	0,295	0,305	0,314	0,323	0,332	0,341	0,351	2
3	0,360	0,369	0,379	0,388	0,397	0,406	0,415	0,424	0,433	0,442	3
4	0,452	0,461	0,470	0,479	0,488	0,497	0,507	0,516	0,525	0,534	4
5	0,543	0,553	0,562	0,571	0,580	0,589	0,598	0,607	0,616	0,626	5
6	0,635	0,644	0,654	0,663	0,672	0,681	0,690	0,699	0,708	0,718	6
7	0,727	0,736	0,745	0,754	0,763	0,772	0,781	0,790	0,799	0,809	7
8	0,818	0,827	0,836	0,845	0,855	0,864	0,873	0,882	0,891	0,900	8
9	0,910	0,919	0,928	0,937	0,946	0,956	0,965	0,974	0,983	0,992	9
10	1,002										

u_{200} m/sec	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	u_{200} m/sec
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Read for the comma a decimal point

Table 7B. Values of $0.485 \times 0.35 (0.5 + 0.54 u)$ for u ($m \text{ sec}^{-1}$) measured at 10 m

u_{1000} m/sec	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	u_{1000} m/sec
0	0,085	0,092	0,098	0,105	0,112	0,119	0,126	0,132	0,139	0,146	0
1	0,153	0,160	0,166	0,173	0,180	0,187	0,193	0,200	0,207	0,214	1
2	0,221	0,227	0,234	0,241	0,248	0,255	0,261	0,268	0,275	0,282	2
3	0,288	0,295	0,302	0,309	0,316	0,322	0,329	0,336	0,343	0,349	3
4	0,356	0,363	0,370	0,377	0,383	0,390	0,397	0,404	0,411	0,417	4
5	0,424	0,431	0,438	0,444	0,451	0,458	0,465	0,472	0,478	0,485	5
6	0,492	0,499	0,505	0,512	0,519	0,526	0,533	0,539	0,546	0,553	6
7	0,560	0,567	0,573	0,580	0,587	0,594	0,600	0,607	0,614	0,621	7
8	0,628	0,634	0,641	0,648	0,655	0,661	0,668	0,675	0,682	0,689	8
9	0,695	0,702	0,709	0,716	0,722	0,729	0,736	0,743	0,750	0,756	9
10	0,763										

u_{1000} m/sec	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	u_{1000} m/sec
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Read for the comma a decimal point

Table 9. Daylength (N) in hours
(Smithsonian Meteorological Tables, table 171)

N	month 10 d.	Jan.			Feb.			March			April			May			June					
		I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III			
0	10 d. month	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12		
5	10 d. month	11,85	11,87	11,88	11,93	11,96	12,00	12,04	12,08	12,13	12,18	12,22	12,27	12,32	12,35	12,37	12,40	12,41	12,42	12,41		
10	10 d. month	11,56	11,61	11,66	11,73	11,81	11,90	11,97	12,06	12,15	12,27	12,35	12,43	12,52	12,57	12,63	12,68	12,70	12,72	12,70		
15	10 d. month	11,28	11,34	11,42	11,53	11,66	11,78	11,90	12,04	12,18	12,33	12,47	12,60	12,72	12,83	12,91	12,98	13,01	13,02	13,00		
20	10 d. month	10,97	11,06	11,17	11,33	11,50	11,67	11,82	12,01	12,20	12,42	12,60	12,77	12,93	13,08	13,20	13,28	13,33	13,35	13,32		
25	10 d. month	10,65	10,76	10,92	11,11	11,33	11,55	11,75	11,98	12,23	12,50	12,74	12,97	13,17	13,35	13,50	13,62	13,68	13,68	13,66		
30	10 d. month	10,30	10,42	10,63	10,88	11,13	11,42	11,67	11,97	12,27	12,58	12,88	12,17	13,44	13,66	13,85	13,99	14,07	14,08	14,05		
35	10 d. month	9,90	10,07	10,30	10,62	10,95	11,28	11,58	11,97	12,30	12,70	13,07	13,40	13,71	14,00	14,23	14,41	14,50	14,50	14,47		
40	10 d. month	9,45	9,65	9,93	10,32	10,70	11,12	11,47	11,92	12,35	12,83	13,25	13,66	14,04	14,38	14,67	14,88	15,00	15,00	14,96		
42	10 d. month	9,25	9,45	9,78	10,19	10,60	11,05	11,42	11,90	12,37	12,88	13,35	13,78	14,20	14,55	14,85	15,10	15,22	15,23	15,18		
44	10 d. month	9,02	9,25	9,60	10,04	10,49	10,97	11,37	11,86	12,39	12,94	13,43	13,92	14,35	14,74	15,07	15,33	15,47	15,47	15,42		
46	10 d. month	8,78	9,05	9,42	9,89	10,38	10,90	11,32	11,87	12,42	13,00	13,53	14,04	14,52	14,95	15,29	15,58	15,68	15,75	15,67		
48	10 d. month	8,54	8,82	9,21	9,73	10,25	10,82	11,27	11,85	12,43	13,07	13,65	14,19	14,71	15,17	15,54	15,86	16,02	16,03	15,97		
50	10 d. month	8,26	8,57	8,99	9,55	10,12	10,72	11,21	11,83	12,47	13,16	13,76	14,34	14,90	15,41	15,82	16,17	16,35	16,35	16,29		
52	10 d. month	7,95	8,27	8,75	9,37	9,97	10,61	11,14	11,83	12,49	13,22	13,89	14,52	15,12	15,67	16,13	16,51	16,70	16,72	16,64		
54	10 d. month	7,60	7,96	8,47	9,14	9,81	10,52	11,08	11,80	12,53	13,32	14,02	14,72	15,37	15,97	16,47	16,89	17,11	17,12	17,04		
56	10 d. month	7,12	7,61	8,17	8,91	9,63	10,39	11,00	11,78	12,57	13,42	14,18	14,94	15,65	16,47	16,87	17,35	17,57	17,60	17,51		
58	10 d. month	6,74	7,21	7,80	8,63	9,42	10,25	10,92	11,77	12,60	13,52	14,35	15,17	15,96	16,70	17,32	17,86	18,13	18,15	18,05		
60	10 d. month	6,22	6,72	7,44	8,33	9,20	10,10	10,82	11,73	12,66	13,65	14,57	15,44	16,31	17,12	17,96	18,47	18,80	18,83	18,70		
61	10 d. month	5,90	6,46	7,22	8,16	9,07	10,02	10,77	11,72	12,67	13,71	14,67	15,60	16,52	17,40	18,17	18,85	19,22	19,22	19,10		
62	10 d. month	5,53	6,16	6,97	7,98	8,93	9,93	10,72	11,71	12,70	13,78	14,78	15,77	16,73	17,67	18,53	19,27	19,68	19,69	19,55		
63	10 d. month	5,13	5,82	6,70	7,78	8,80	9,83	10,65	11,68	12,73	13,87	14,90	15,93	16,97	17,98	18,90	19,77	20,22	20,25	20,08		
64	10 d. month	4,68	5,45	6,40	7,55	8,63	9,73	10,60	11,67	12,77	13,95	15,03	16,13	17,23	18,33	19,37	20,35	20,92	20,98	20,75		
65	10 d. month	4,15	5,05	6,08	7,32	8,47	9,62	10,53	11,67	12,80	14,03	15,18	16,35	17,53	18,72	19,88	21,08	21,87	21,97	21,64		
0	10 d. month	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I		
		July			August			Sept.			Oct.			Nov.			Dec.					

Evapotranspiration

(Table 9 cont.)

July			August			Sept.			Oct.			Nov.			Dec.		
I	II	III	I	II	III	I	II	III	I	II	III	I	II	III	I	II	III
12,12	12,12	12,12	12,11	12,10	12,10	12,11	12,11	12,11	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,12	12,13
	12,12		12,10			12,11			12,12			12,12			12,12		12,12
12,40	12,38	12,36	12,33	12,28	12,23	12,19	12,16	12,10	12,05	12,02	11,97	11,92	11,90	11,87	11,85	11,83	11,83
	12,38		12,28			12,16			12,02			11,90			11,83		11,83
12,68	12,64	12,60	12,53	12,45	12,37	12,27	12,17	12,08	12,00	11,91	11,83	11,75	11,67	11,61	11,58	11,54	11,54
	12,64		12,45			12,17			11,91			11,67			11,55		11,55
12,98	12,94	12,85	12,73	12,62	12,50	12,35	12,22	12,08	11,95	11,82	11,67	11,55	11,43	11,35	11,28	11,22	11,24
	12,92		12,62			12,22			11,81			11,44			11,25		11,25
13,30	13,23	13,12	12,97	12,83	12,62	12,45	12,25	12,07	11,88	11,70	11,52	11,35	11,19	11,07	10,97	10,93	10,93
	13,22		12,81			12,26			11,70			11,20			10,94		10,94
13,63	13,56	13,41	13,22	13,02	12,82	12,55	12,31	12,07	11,82	11,58	11,35	11,12	10,93	10,78	10,67	10,60	10,60
	13,53		13,02			12,31			11,58			10,94			10,62		10,62
14,02	13,89	13,72	13,49	13,23	12,96	12,65	12,34	12,05	11,76	11,47	11,18	10,89	10,65	10,47	10,32	10,23	10,22
	13,88		13,23			12,35			11,47			10,67			10,26		10,26
14,45	14,29	14,07	13,78	13,48	13,16	12,77	12,42	12,07	11,68	11,33	10,98	10,63	10,34	10,10	9,93	9,82	9,82
	14,27		13,47			12,42			11,33			10,36			9,86		9,86
14,92	14,74	14,48	14,13	13,77	13,37	12,91	12,48	12,06	11,60	11,18	10,77	10,33	9,98	9,70	9,48	9,35	9,35
	14,71		13,76			12,48			11,18			10,00			9,39		9,39
15,12	14,92	14,67	14,29	13,90	13,46	12,97	12,51	12,06	11,57	11,12	10,67	10,22	9,83	9,52	9,29	9,15	9,15
	14,90		13,88			12,51			11,12			9,86			9,20		9,20
15,37	15,17	14,87	14,44	14,02	13,57	13,03	12,54	12,03	11,54	11,04	10,57	10,07	9,67	9,32	9,07	8,92	8,92
	15,14		14,01			12,53			11,03			9,69			8,97		8,97
15,62	15,40	15,07	14,62	14,17	13,68	13,11	12,57	12,04	11,51	10,97	10,45	9,92	9,48	9,11	8,83	8,68	8,65
	15,36		14,16			12,57			10,98			9,50			8,72		8,72
15,91	15,66	15,30	14,82	14,32	13,80	13,18	12,61	12,03	11,46	10,88	10,33	9,75	9,28	8,86	8,57	8,41	8,37
	15,62		14,31			12,61			10,89			9,30			8,45		8,45
16,22	15,95	15,55	15,03	14,50	13,92	13,27	12,67	12,03	11,41	10,80	10,21	9,58	9,07	8,63	8,30	8,12	8,09
	15,91		14,48			12,66			10,80			9,07			8,17		8,17
16,56	16,27	15,83	15,26	14,67	14,06	13,35	12,69	12,02	11,37	10,70	10,07	9,39	8,82	8,37	8,00	7,82	7,77
	16,22		14,66			12,69			10,71			8,86			7,86		7,86
16,95	16,63	16,15	15,52	14,88	14,21	13,44	12,74	12,02	11,32	10,60	9,91	9,17	8,56	8,05	7,66	7,42	7,40
	16,58		14,87			12,73			10,61			8,59			7,49		7,49
17,40	17,04	16,50	15,81	15,12	14,38	13,55	12,81	12,02	11,26	10,48	9,74	8,95	8,26	7,71	7,26	7,00	6,97
	16,98		15,10			12,79			10,49			8,31			7,08		7,08
17,92	17,52	16,91	16,15	15,38	14,57	13,67	12,85	12,02	11,20	10,37	9,54	8,68	7,93	7,32	6,81	6,52	6,48
	17,45		15,37			12,85			10,37			7,98			6,60		6,60
18,57	18,07	17,38	16,52	15,67	14,81	13,81	12,92	12,02	11,12	10,22	9,33	8,38	7,57	6,85	6,28	5,93	5,90
	18,01		15,67			12,92			10,22			7,60			6,04		6,04
18,95	18,40	17,67	16,73	15,83	14,91	13,89	12,93	12,02	11,07	10,15	9,22	8,21	7,34	6,60	5,98	5,61	5,57
	18,34		15,82			12,95			10,15			7,38			5,72		5,72
19,40	18,77	17,98	16,97	16,02	15,05	13,97	13,00	11,98	11,03	10,05	9,08	8,03	7,12	6,30	5,62	5,22	5,18
	18,72		16,01			12,98			10,05			7,15			5,34		5,34
19,90	19,19	18,30	17,23	16,21	15,25	14,05	13,03	12,00	11,00	9,97	8,95	7,83	6,87	5,97	5,25	4,80	4,73
	19,13		16,23			13,03			9,97			6,89			4,93		4,93
20,52	19,67	18,68	17,50	16,42	15,33	14,15	13,08	12,00	10,95	9,87	8,80	7,63	6,58	5,62	4,82	4,30	4,27
	19,62		16,42			13,08			9,87			6,61			4,46		4,46
21,30	20,16	19,10	17,82	16,65	15,50	14,25	13,12	12,00	10,90	9,77	8,65	7,40	6,27	5,22	4,28	3,71	3,65
	20,22		16,66			13,12			9,77			6,30			3,88		3,88

Round from the summa a decimal point

Table 8. Saturated vapour pressure (e_{sat}).

(from: Handbook of Chemistry and Physics, 49th Ed., pp.D-109)

T_2	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9	T_2
-10	2,15										-10
-9	2,32	2,30	2,29	2,27	2,26	2,24	2,22	2,21	2,19	2,17	-9
-8	2,51	2,49	2,47	2,45	2,43	2,41	2,40	2,38	2,36	2,34	-8
-7	2,71	2,69	2,67	2,65	2,63	2,61	2,59	2,57	2,55	2,53	-7
-6	2,93	2,91	2,89	2,86	2,84	2,82	2,80	2,77	2,75	2,73	-6
-5	3,16	3,14	3,11	3,09	3,06	3,04	3,01	2,99	2,97	2,95	-5
-4	3,41	3,39	3,37	3,34	3,32	3,29	3,27	3,24	2,22	3,18	-4
-3	3,67	3,64	3,62	3,59	3,57	3,54	3,51	3,49	3,46	3,44	-3
-2	3,96	3,93	3,90	3,87	3,84	3,82	3,79	3,76	3,73	3,70	-2
-1	4,26	4,23	4,20	4,17	4,14	4,11	4,08	4,05	4,02	4,00	-1
-0	4,58	4,55	4,51	4,48	4,45	4,42	4,39	4,36	4,32	4,29	-0
0	4,58	4,62	4,65	4,69	4,71	4,75	4,78	4,82	4,86	4,89	0
1	4,93	4,96	5,00	5,03	5,07	5,11	5,14	5,18	5,22	5,25	1
2	5,29	5,33	5,37	5,41	5,45	5,49	5,53	5,57	5,60	5,64	2
3	5,68	5,72	5,77	5,81	5,85	5,89	5,93	5,97	6,01	6,06	3
4	6,10	6,14	6,19	6,23	6,27	6,31	6,36	6,40	6,45	6,49	4
5	6,54	6,58	6,64	6,68	6,73	6,77	6,82	6,87	6,92	6,96	5
6	7,01	7,06	7,11	7,16	7,21	7,25	7,31	7,36	7,41	7,46	6
7	7,51	7,56	7,62	7,67	7,72	7,77	7,83	7,88	7,94	7,98	7
8	8,04	8,10	8,15	8,21	8,27	8,32	8,38	8,43	8,49	8,54	8
9	8,61	8,67	8,73	8,78	8,84	8,90	8,96	9,02	9,09	9,14	9
10	9,21	9,26	9,33	9,39	9,46	9,52	9,58	9,65	9,71	9,77	10
11	9,84	9,91	9,90	10,04	10,11	10,17	10,24	10,31	10,38	10,45	11
12	10,52	10,58	10,66	10,72	10,80	10,87	10,94	11,00	11,08	11,15	12
13	11,23	11,30	11,38	11,45	11,53	11,60	11,68	11,76	11,83	11,91	13
14	11,99	12,06	12,14	12,22	12,30	12,38	12,46	12,54	12,62	12,70	14
15	12,79	12,86	12,95	13,03	13,12	13,20	13,29	13,37	13,46	13,54	15
16	13,63	13,72	13,81	13,90	13,99	14,08	14,17	14,26	14,35	14,44	16
17	14,53	14,62	14,71	14,80	14,90	14,99	15,09	15,18	15,28	15,38	17
18	15,48	15,57	15,67	15,77	15,87	15,97	16,07	16,17	16,27	16,37	18
19	16,48	16,58	16,68	16,79	16,89	17,00	17,10	17,21	17,32	17,43	19
20	17,53	17,64	17,75	17,86	17,97	18,08	18,20	18,31	18,42	18,54	20
21	18,65	18,77	18,88	19,00	19,11	19,23	19,35	19,47	19,59	19,71	21
22	19,83	19,95	20,07	20,19	20,32	20,44	20,57	20,69	20,82	20,93	22
23	21,07	21,19	21,32	21,45	21,58	21,71	21,84	21,97	22,11	22,24	23
24	22,38	22,51	22,65	22,78	22,92	23,06	23,20	23,34	23,48	23,62	24
25	23,76	23,90	24,04	24,18	24,33	24,47	24,62	24,76	24,91	25,06	25
26	25,21	25,36	25,51	25,66	25,81	25,96	26,12	26,27	26,43	26,58	26
27	26,74	26,90	27,06	27,21	27,37	27,53	27,70	27,86	28,02	28,18	27
28	28,35	28,51	28,68	28,85	29,02	29,18	29,35	29,52	29,70	29,87	28
29	30,04	30,21	30,39	30,56	30,74	30,92	31,10	31,28	31,46	31,64	29
30	31,82	32,00	32,19	32,37	32,56	32,74	32,93	33,12	33,31	33,50	30
31	33,70	33,89	34,08	34,28	34,47	34,66	34,86	35,06	35,26	35,46	31
32	35,66	35,86	36,07	36,27	36,48	36,68	36,89	37,10	37,31	37,52	32
33	37,73	37,94	38,16	38,37	38,58	38,80	39,02	39,24	39,46	39,68	33
34	39,90	40,12	40,34	40,57	40,80	41,02	41,25	41,48	41,71	41,94	34
35	42,18	42,41	42,64	42,88	43,12	43,36	43,60	43,84	44,08	44,32	35
36	44,56	44,80	45,05	45,30	45,55	45,80	46,05	46,30	46,56	46,81	36
37	47,07	47,32	47,58	47,84	48,10	48,36	48,63	48,89	49,16	49,42	37
38	49,69	49,96	50,28	50,50	50,77	51,04	51,32	51,60	51,88	52,16	38
39	52,44	52,72	53,01	53,29	53,58	53,87	54,16	54,45	54,74	55,08	39
40	55,32	55,61	55,91	56,21	56,51	56,81	57,11	57,41	57,72	58,03	40

Read for the comma a decimal point

Table 10. Value of *i* (Thorntwaite's monthly heat index)

T°C	,0	,1	,2	,3	,4	,5	,6	,7	,8	,9
0			,01	,01	,02	,03	,04	,05	,06	,07
1	,09	,10	,12	,13	,15	,16	,18	,20	,21	,23
2	,25	,27	,29	,31	,33	,35	,37	,39	,42	,44
3	,46	,48	,51	,53	,56	,58	,61	,63	,66	,69
4	,71	,74	,77	,80	,82	,85	,88	,91	,94	,97
5	1,00	1,03	1,06	1,09	1,12	1,16	1,19	1,22	1,25	1,29
6	1,32	1,35	1,39	1,42	1,45	1,49	1,52	1,56	1,59	1,63
7	1,66	1,70	1,74	1,77	1,81	1,85	1,89	1,92	1,96	2,00
8	2,04	2,08	2,12	2,15	2,19	2,23	2,27	2,31	2,35	2,39
9	2,44	2,48	2,52	2,56	2,60	2,64	2,69	2,73	2,77	2,81
10	2,86	2,90	2,94	2,99	3,03	3,08	3,12	3,16	3,21	3,25
11	3,30	3,34	3,39	3,44	3,48	3,53	3,58	3,62	3,67	3,72
12	3,76	3,81	3,86	3,91	3,96	4,00	4,05	4,10	4,15	4,20
13	4,25	4,30	4,35	4,40	4,45	4,50	4,55	4,60	4,65	4,70
14	4,75	4,81	4,86	4,91	4,96	5,01	5,07	5,12	5,17	5,22
15	5,28	5,33	5,38	5,44	5,49	5,55	5,60	5,65	5,71	5,76
16	5,82	5,87	5,93	5,98	6,04	6,10	6,15	6,21	6,26	6,32
17	6,38	6,44	6,49	6,55	6,61	6,66	6,72	6,78	6,84	6,90
18	6,95	7,01	7,07	7,13	7,19	7,25	7,31	7,37	7,43	7,49
19	7,55	7,61	7,67	7,73	7,79	7,85	7,91	7,97	8,03	8,10
20	8,16	8,22	8,28	8,34	8,41	8,47	8,53	8,59	8,66	8,72
21	8,78	8,85	8,91	8,97	9,04	9,10	9,17	9,23	9,29	9,36
22	9,42	9,49	9,55	9,62	9,68	9,68	9,75	9,82	9,95	10,01
23	10,08	10,15	10,21	10,28	10,35	10,41	10,48	10,55	10,62	10,68
24	10,75	10,82	10,89	10,95	11,02	11,09	11,16	11,23	11,30	11,37
25	11,44	11,50	11,57	11,64	11,71	11,78	11,85	11,92	11,99	12,06
26	12,13	12,21	12,28	12,35	12,42	12,49	12,56	12,63	12,70	12,78
27	12,85	12,92	12,99	13,07	13,14	13,21	13,28	13,36	13,43	13,50
28	13,58	13,65	13,72	13,80	13,87	13,94	14,02	14,09	14,17	14,24
29	14,32	14,39	14,47	14,54	14,62	14,69	14,77	14,84	14,92	14,99
30	15,07	15,15	15,22	15,30	15,38	15,45	15,53	15,61	15,68	15,76
31	15,84	15,92	15,99	16,07	16,15	16,23	16,30	16,38	16,46	16,54
32	16,62	16,70	16,78	16,85	16,93	17,01	17,09	17,17	17,25	17,33
33	17,41	17,49	17,57	17,65	17,73	17,81	17,89	17,97	18,05	18,13
34	18,22	18,30	18,38	18,46	18,54	18,62	18,70	18,79	18,87	18,95
35	19,03	19,11	19,20	19,28	19,36	19,45	19,53	19,61	19,69	19,78
36	19,86	19,95	20,03	20,11	20,20	20,28	20,36	20,45	20,53	20,62
37	20,70	20,79	20,87	20,96	21,04	21,13	21,21	21,30	21,38	21,47
38	21,56	21,64	21,73	21,81	21,90	21,99	22,07	22,16	22,25	22,33
39	22,42	22,51	22,59	22,68	22,77	22,86	22,95	23,03	23,12	23,21
40	23,30									

Read for the comma a decimal point

Table 11. Mean possible duration of sunlight in the Northern and Southern hemispheres expressed in units of 30 days of 12 hours each

month/ latitude	J	F	M	A	M	J	J	A	S	O	N	D
$^{\circ}_N$												
0	1,04	,94	1,04	1,01	1,04	1,01	1,04	1,04	1,01	1,04	1,01	1,04
5	1,02	,93	1,03	1,02	1,06	1,03	1,06	1,05	1,01	1,03	,99	1,02
10	1,00	,91	1,03	1,03	1,08	1,06	1,08	1,07	1,02	1,02	,98	,99
15	,97	,91	1,03	1,04	1,11	1,08	1,12	1,08	1,02	1,01	,95	,97
20	,95	,90	1,03	1,05	1,13	1,11	1,14	1,11	1,02	1,00	,93	,94
25	,93	,89	1,03	1,06	1,15	1,14	1,17	1,12	1,02	,99	,91	,91
26	,92	,88	1,03	1,06	1,15	1,15	1,17	1,12	1,02	,99	,91	,91
27	,92	,88	1,03	1,07	1,16	1,15	1,18	1,13	1,02	,99	,90	,90
28	,91	,88	1,03	1,07	1,16	1,16	1,18	1,13	1,02	,98	,90	,90
29	,91	,87	1,03	1,07	1,17	1,16	1,19	1,13	1,03	,98	,90	,89
30	,90	,87	1,03	1,08	1,18	1,17	1,20	1,14	1,03	,98	,89	,88
31	,90	,87	1,03	1,08	1,18	1,18	1,20	1,14	1,03	,98	,89	,88
32	,89	,86	1,03	1,08	1,19	1,19	1,21	1,15	1,03	,98	,88	,87
33	,88	,86	1,03	1,09	1,19	1,20	1,22	1,15	1,03	,97	,88	,86
34	,88	,85	1,03	1,09	1,20	1,20	1,22	1,16	1,03	,97	,87	,86
35	,87	,85	1,03	1,09	1,21	1,21	1,23	1,16	1,03	,97	,86	,85
36	,87	,85	1,03	1,10	1,21	1,22	1,24	1,16	1,03	,97	,86	,84
37	,86	,84	1,03	1,10	1,22	1,23	1,25	1,17	1,03	,97	,85	,83
38	,85	,84	1,03	1,10	1,23	1,24	1,25	1,17	1,04	,96	,84	,83
39	,85	,84	1,03	1,11	1,23	1,24	1,26	1,18	1,04	,96	,84	,82
40	,84	,83	1,03	1,11	1,24	1,25	1,27	1,18	1,04	,96	,83	,81
41	,83	,83	1,03	1,11	1,25	1,26	1,27	1,19	1,04	,96	,82	,80
42	,82	,83	1,03	1,12	1,26	1,27	1,28	1,19	1,04	,95	,82	,79
43	,81	,82	1,02	1,12	1,26	1,28	1,29	1,20	1,04	,95	,81	,77
44	,81	,82	1,02	1,13	1,27	1,29	1,30	1,20	1,04	,95	,80	,76
45	,80	,81	1,02	1,13	1,28	1,29	1,31	1,21	1,04	,94	,79	,75
46	,79	,81	1,02	1,13	1,29	1,31	1,32	1,22	1,04	,94	,79	,74
47	,77	,80	1,02	1,14	1,30	1,32	1,33	1,22	1,04	,93	,78	,73
48	,76	,80	1,02	1,14	1,31	1,33	1,34	1,23	1,05	,93	,77	,72
49	,75	,79	1,02	1,14	1,32	1,34	1,35	1,24	1,05	,93	,76	,71
50	,74	,78	1,02	1,15	1,33	1,36	1,37	1,25	1,06	,92	,76	,70
$^{\circ}_S$												
5	1,06	,95	1,04	1,00	1,02	,99	1,02	1,03	1,00	1,05	1,03	1,06
10	1,08	,97	1,05	,99	1,01	,96	1,00	1,01	1,00	1,06	1,05	1,10
15	1,12	,98	1,05	,98	,98	,94	,97	1,00	1,00	1,07	1,07	1,12
20	1,14	1,00	1,05	,97	,96	,91	,95	,99	1,00	1,08	1,09	1,15
25	1,17	1,01	1,05	,96	,94	,88	,93	,98	1,00	1,10	1,11	1,18
30	1,20	1,03	1,06	,95	,92	,85	,90	,96	1,00	1,12	1,14	1,21
35	1,23	1,04	1,06	,94	,89	,82	,87	,94	1,00	1,13	1,17	1,25
40	1,27	1,06	1,07	,93	,86	,78	,84	,92	1,00	1,15	1,20	1,29
42	1,28	1,07	1,07	,92	,85	,76	,82	,92	1,00	1,16	1,22	1,31
44	1,30	1,08	1,07	,92	,83	,74	,81	,91	,99	1,17	1,23	1,33
46	1,32	1,10	1,07	,91	,82	,72	,79	,90	,99	1,17	1,25	1,35
48	1,34	1,11	1,08	,90	,80	,70	,76	,89	,99	1,18	1,27	1,37
50	1,37	1,12	1,08	,89	,77	,67	,74	,88	,99	1,19	1,29	1,41

Read for the comma a decimal point

Table 12. The uncorrected potential evapotranspiration, according to Thornthwaite's formula, for temperatures greater than 26.5 °C.

Temperature T (°C)	E_p (cm.month ⁻¹) uncorrected value
26,5	13,50
27,0	13,95
27,5	14,37
28,0	14,78
28,5	15,17
29,0	15,54
29,5	15,89
30,0	16,21
30,5	16,52
31,0	16,80
31,5	17,07
32,0	17,31
32,5	17,53
33,0	17,72
33,5	17,90
34,0	18,05
34,5	18,18
35,0	18,29
35,5	18,37
36,0	18,43
36,5	18,47
37,0	18,49
37,5	18,50
38,0	18,50

Read for the comma a decimal point

SURVEYS AND INVESTIGATIONS

20. HYDROPEDOLOGICAL SURVEY

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PURPOSE AND SCOPE

A discussion of the principal hydrological soil properties to be determined in analyzing and evaluating an area's drainage problems and in planning and designing an adequate drainage system.

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20.1 INTRODUCTION

It is the aim of this chapter to define the contribution that a soil scientist can make to the basic information required to analyse and evaluate an area's drainage problems and to plan and design an adequate drainage system. The first question to be answered in any drainage study is what is the soil potential of the area? A clear statement must be made on whether or not the soils in the area are suitable for growing the proposed crops. An inventory of the soil resources should therefore be made, applying modern techniques of soil and land classification mapping.

The conventional soil or land classification maps produced in this way, however, have only a limited value for drainage purposes because they provide no information on the layers below 1.20 or 1.50 m. The quantitative hydrological or physical data required for drainage must be supplied by supplementary investigations that extend to a depth of 4 to 5 m below ground surface and even deeper. These investigations can be grouped under the heading of a hydropedological survey.

A hydropedological survey should throw light on such questions as:

- where do excesses of surface water occur and what are the probable causes?
- which areas suffer from unfavourable groundwater conditions and what is causing these conditions?
- is excessive wetness of the soils interfering with tillage, crop growth, and harvesting?
- which areas suffer from unfavourable soil conditions, e.g. salinity, and what are the relationships, if any, between such conditions and local surface and groundwater conditions? Can these unfavourable conditions be rectified?
- what properties do the soils of the area possess that will hinder or promote the flow of water through them and will decide whether it is an economic proposition to control the groundwater table?
- what are the main topographical, physiographical, and relief features of the area? Will the disposal of excess surface water or drain water be possible and what are the conditions of the outlet?
- what effect will artificial drainage have on the soils and on their productivity?

TABLE 1. Soil and land features relevant to drainage.
 This table is based to a large extent on unpublished data of VAN BEERS (1973)

MAIN ASPECTS	COMPONENTS	PROPERTY TO BE CHARACTERIZED OR PREDICTED	DEPTH BEING CONSIDERED (m)	REQUIRED OBSERVATIONS, OR FIELD MEASUREMENTS, AND RELATED USEFUL SOIL PROPERTIES FOR INTERPRETATION
SOIL DRAINAGE (vertical flow generally)	Infiltration (surface intake)	Cumulative intake; Instantaneous intake rate	0 - 0.3 upper rootzone mainly	<ul style="list-style-type: none"> • Infiltrometer tests (dry run); • Field estimates of soil texture; Swelling behaviour of clays; Organic matter content; Presence of free carbonates; Gypsum crystals; Salt crusts; Soil structure and structure stability (soil aggregate dispersion in water, soil crusts); Soil pH; Soil colour; Consistence; Visible pores, cracks, root density
	Internal drainage (soil profile drainage)	Percolation rate	0.3 - 1.2 Lower rootzone	<ul style="list-style-type: none"> • Presence of layers with limiting permeability (depth, thickness). • Infiltrometer tests (wet run); • Field estimates on basis of soil properties (see above) and presence of clay skins; Penetration depth and distribution of roots.
	Water storage above field capacity	Actual or potential perched water table or groundwater level, incl. ave. seasonal fluctuations; Groundwater salinity		<ul style="list-style-type: none"> • Monitoring of groundwater levels in shallow wells; • Groundwater quality (EC,SAR); • Soil colour and gley phenomena; Iron and/or manganese concretions (type, amount and depth of occurrence).
		Storage capacity (drainable pore space)		<ul style="list-style-type: none"> • Vertical soil moisture content distribution during high and low groundwater levels; • Laboratory determination of pF curve on undisturbed core samples; • Field estimates on basis of texture, structure, etc.

	Surface drainage	Land surface features influencing surface drainage; Erosion hazards	Land surface <i>sway</i>	
LAND DRAINAGE (lateral flow generally)	Surface drainage	<ul style="list-style-type: none"> Slope (degree and length); Vegetative cover; Natural stream channels (distribution, size, depth, gradient); Interference with stream regimen or overland flow (channel obstructions, need for channel improvements, fences, roads, need for land grading); Size distribution of bed load as indicator of stream regimen. Natural outfall conditions; Soil structure and structure stability. 	<ul style="list-style-type: none"> Presence and depth of an impervious layer, hardpan, stones, unstable very fine sandy or silty layers, muddy or peaty deposits; Hydraulic conductivity tests; Hydraulic conductivity estimates on basis of correlations with particle size (sandy layers) or structure and pores (loamy or clayey layers) for main distinct water conducting strata; Depth and quality of groundwater (EC, SAR); Vertical changes in soil texture promoting or impeding upward capillary transport of groundwater and - with respect to arid climates - dissolved salts. 	
	Subsurface drainage	<ul style="list-style-type: none"> Drainability of shallow substratum 	<ul style="list-style-type: none"> 1.2 - 2.0 <i>shallow substratum</i> 	
		<ul style="list-style-type: none"> Drainability of medium deep substratum 	<ul style="list-style-type: none"> 2.0 - 5.0 <i>medium deep substratum</i> 	
		Geohydrological characteristics	> 5.0 <i>deep substratum</i>	<ul style="list-style-type: none"> Augerings only at selected strategic locations recording soil texture of substrata; Depth to impervious layer, if any, and hydraulic conductivity above this layer; Transmissibility (KD values); Depth and quality (EC, SAR) of groundwater; Piezometric surface of groundwater. Geohydrological investigations.
FUTURE DEVELOPMENT OF DRAINAGE CONDITIONS	Future drain- and ditch levels	Subsidence	0 - 5.0	<ul style="list-style-type: none"> Presence of muddy and peaty deposits (thickness, water content, organic matter %, soil texture parameters); Outfall conditions.
	Hydraulic conductivity changes	Soil "ripening"	0 - 2.0	<ul style="list-style-type: none"> Degree of soil ripening (crack and biopore development, aeration mottles, irreversible moisture losses); Hydraulic conductivity (k_i) test; Experimental field research.
		Effects of cropping and desalinization		0 - 1.2

1 The minimum depth of routine soil investigations for all irrigation and drainage projects is 2 m

20.2 REQUIRED SOIL DATA AND MAPS

In drainage, a distinction must be made between soil drainage and land drainage. By soil drainage we mean the ability of the soil to transmit water in a direction that is mainly vertical. This will take place mainly in the upper 1.20 m of the soil profile. The components of soil drainage are infiltration, internal drainage, and water storage above field capacity.

Land drainage covers both surface drainage and subsurface drainage, in both of which the flow of water is mainly lateral. When speaking of subsurface drainage, we are usually only interested in that taking place in the upper 5 m of the soil profile, but sometimes a considerably greater depth is involved.

Each type of drainage survey requires its own set of investigations, although some observations and measurements are common to all of them. Table 1 lists the main soil and land features relevant to drainage and the observations and measurements required to determine these properties. The task of a soil scientist working in a drainage project can be derived from a study of this table.

It is not the intention of this chapter to discuss each item of this list; many of them are self-explanatory while others have been dealt with in detail in other chapters. In the following pages, only some of the important subjects which have not yet been covered in other chapters will be discussed.

But first, some remarks on maps and mapping techniques.

Maps are the basis as well as the end result of a hydropedological survey. Hence, before field work is started, all existing maps of the project area and its surroundings should be collected, studied, and, if necessary, transformed. Such maps include topographical, geological, physiographical, soil, and land classification maps, and also air photographs. The mapping scale depends on the type of survey being conducted: reconnaissance, semi-detailed, or detailed. In conventional soil surveys, one can distinguish various mapping intensities, there being a close relationship between survey intensity, mapping scale, and level of mapping units (see Table 2).

The soil and topographical maps which are used as the basis for a hydropedological survey are preferably at scales 1:5,000 or 1:10,000.

As already mentioned, conventional soil maps have limited value in land drainage, especially if a subsurface drainage system is to be installed. The shortcomings of such maps are listed in Table 3.

TABLE 2. Types of soil surveys (after BURINGH, STEUR, and VINK, 1962)¹

Type of soil map	Area of smallest mappable soil unit in ha	Possible observation density ² obs./ha	Average scale of final map	Variations in scale of map
Special soil map	< 0.1	2 per ha	1:5,000	larger than 1:7,500
Detailed soil map	0.1 - 0.5	1 per 2 ha	1:10,000	1:7,500 to 1:17,500
Semi-detailed soil map	0.5 - 3.0	1 per 12½ ha	1:25,000	1:17,500 to 1:35,000
Reconnaissance soil map	3.0 - 15	1 per 50 ha	1:50,000	1:35,000 to 1:75,000
General soil map	15 - 60	1 per 200 ha	1:100,000	1:75,000 to 1:250,000
Schematic soil map	> 60	1 per 5000 ha	1:500,000	smaller than 1:250,000

¹ In the Soil Survey Manual (USDA Handbook 18) other figures are mentioned

² These figures are based on an observation density of 1 per 2 cm² on the final map. Actual requirements in drainage or irrigation projects may differ considerably from the guideline given here

TABLE 3. Common shortcomings of conventional soil maps for use in drainage design

Required planning data	Relevant soil characteristics	Possibility of deduction from conventional soil map
1. Localization and estimation of waterlogging. Outlining of the drainage system.	Gley symptoms. Groundwater fluctuations. Available moisture.	Can be partly deduced.
1.1 Potential physical and chemical changes (ripening), including peat decomposition.	Consistency. Weight % mineral parts and volume weight. Water content. Organic matter content.	Can be partly deduced (scale limitation and limitation of observation depth).
1.2 Subsidence.	Compressibility and volume weight of the different soil layers, pore content of peat.	Cannot be deduced (observation depth limited).
2. Desirable ditch water levels and drainage depth.	Available moisture. Hydraulic conductivity. Irreversible drying out.	Can only be partly deduced. Cannot be deduced (observation depth limited). Cannot be fully deduced.
3. Density of the drainage system.	Hydraulic conductivity.	Cannot be fully deduced (observation depth limited).
4. Capacity of drainage system.		
4.1 Water-storing capacity.	Tension-free pore volume, volume above phreatic surface.	Cannot be fully deduced.
4.2 Seepage.	Transmissivity (kD)	Cannot be deduced (observation depth limited).
4.3 Stability of excavation. Permissible gradient of side slopes.	Angle of internal friction, volume weight, cohesion, quicksand.	Cannot be deduced (observation depth limited)
5. Permissible load. Stability and settlement of the foundation basis for structures.	Compressibility. Volume weight. Hydraulic conductivity. Angle of internal friction. Cohesion.	Cannot be deduced (observation depth limited)

Hydropedological survey

The task of a hydropedologist working in land drainage is to produce the following types of maps, most of which are single value maps:

- a soil texture map of the upper 2 to 5 m, as required
- a contour map of the upper side of the impervious base layer
- a map showing the location, extent, and thickness of coarse sandy or gravelly layers at or below drain depth
- a map showing the thickness of the water-transmitting layer or layers
- a soil salinity map
- a depth-to-water-table map
- a water table fluctuation map, based on soil hydromorphic characteristics (gley)
- a groundwater salinity map
- a hydraulic conductivity map for layers above and below the water table, especially at and below drain depth
- a map showing the magnitude and variations in the rate of infiltration
- a land-use and vegetative-cover map.

Obviously, the hydropedologist will not always be called upon to prepare all of these maps, nor will he always be able to. If, for example, the impervious base layer lies deeper than 4 to 5 m below the soil surface, the help of a geologist will be required to produce a contour map of the surface of this layer. If the problem is one of watertable control, a vegetative-cover map will not be needed, whereas in the study of surface drainage problems such a map will be of great value. Similarly, in areas with very thick peat layers a map showing the thickness of the peat can only be made by the hydropedologist if the peat is less than 5 m thick.

It may happen that within the upper 2 to 4 m of the soil profile a hardpan or other type of impeding layer is found. If so, it will be necessary to prepare a map showing the occurrence and lateral extent of such a layer.

The above maps are usually prepared as single value maps, each subject being portrayed on a single map. It is, however, convenient to reduce the number of these maps by combining certain subjects; for example, depth to water table and groundwater salinity can often be portrayed on one map, using contour lines for one subject and hatchings for the other. Similarly, hydraulic conductivity and thickness of the water transmitting layer can be combined in one map on which in a later stage drain discharges, available head, and drain depth can be plotted. Such a map does not need contour lines for each item, plottings which allow the direct calculation of drain spacings being sufficient.

20.3 HYDROPEDOLOGICAL SURVEYS AND INVESTIGATIONS

As can be seen from Tables 1 and 3, the hydro pedological data required for planning and designing drainage systems are many and manifold. In the following pages we will limit our discussion to

- hydromorphic soil properties
- infiltration rates
- structure stability
- percolation rates
- hydraulic conductivity
- soil moisture storage
- soil moisture retention
- salinity and alkalinity
- land subsidence and soil ripening.

20.3.1 HYDROMORPHIC SOIL PROPERTIES

As the morphological development of a soil profile is known to be influenced by many factors (Chap.2, Vol.I), including the groundwater regime, it is logical that certain morphological features will yield information on groundwater conditions. This implies that the morphology of the soil has come to some sort of "equilibrium" with the behaviour of the water table and that the properties of the first truly reflect the peculiarities of the second. This notion does not always hold and the processes leading to an equilibrium state can be very complex. In addition to the groundwater regime, factors like the oxygen content of the groundwater, the chemical and mineralogical composition of the soil horizons, and the soil temperature during saturated conditions play an important role in the morphological development of the soil profile. Hence imprecise (though cheap) information on the behaviour of the water table can be inferred from hydromorphic soil properties.

Hydromorphic characteristics which allow the natural internal drainage conditions of soils to be assessed are:

- Soil colour, especially mottling and gley phenomena, which depend on the presence and oxidation-reduction status of free oxides, primarily iron but also manganese, and on the presence of organic matter. The colour patterns of mottles and matrix are directly related to the pattern of translocation of these colouring agents through the soil mass, and to the differences in accessibility for water and atmospheric oxygen along cracks, pores, and biogenic channels as compared with other parts of the soil mass away from these bigger voids.

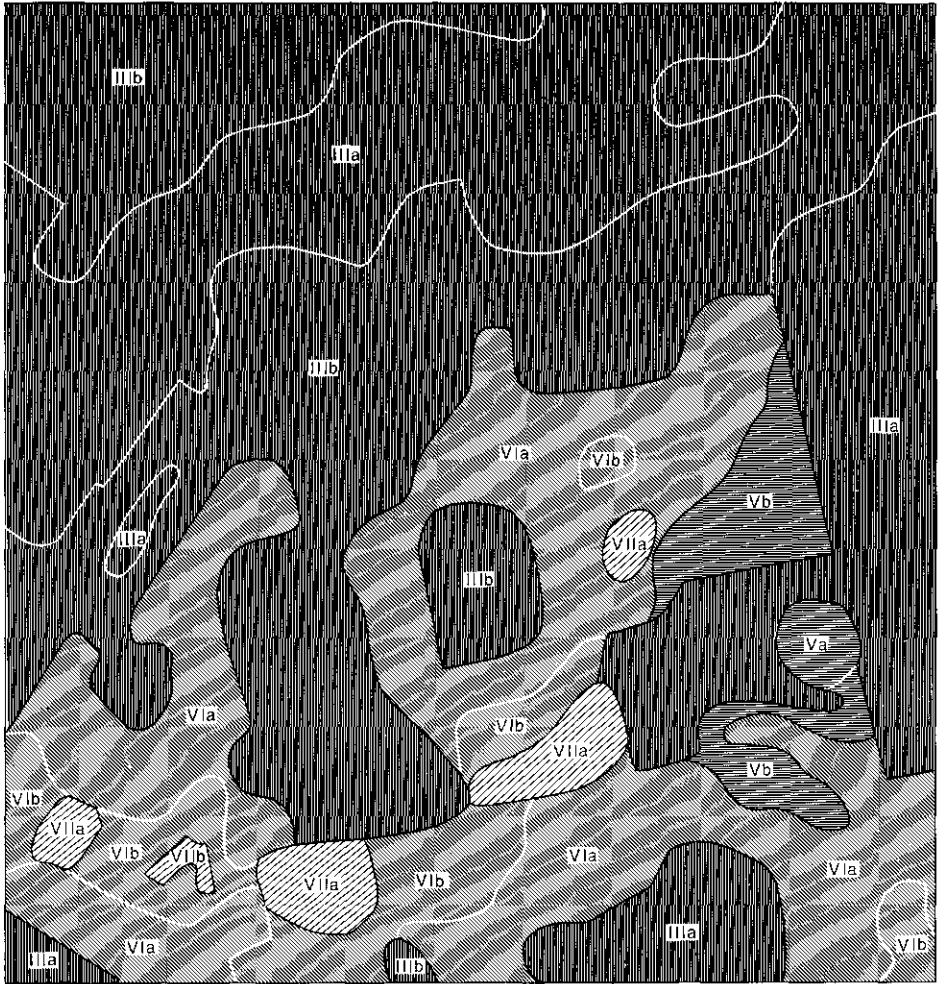
These visible characteristics have long been in use to classify soils in seven relative soil-drainage classes (USDA Soil Survey Manual, 1951).

- Gley soils are grey coloured by active reduction of iron to Fe^{2+} . The peculiar property of this grey colour is that it can change rapidly to a brown colour on exposure to the air. Gley soils do not develop during short periods of waterlogging, say a few weeks, or at very low temperatures, because, like any other chemical reaction, gley formation takes time. Hence the absence of gley phenomena must not be interpreted to mean the absence of waterlogging problems. On the other hand, mottles may remain visible in soils which have been drained for a long time. In dense wet soils these phenomena can remain visible for centuries. It is difficult to determine whether mottles are "fossil" or recent, but if they are observed in the profile it is advisable to auger down to see whether the groundwater table can be reached. Rust coloured mottles may also develop in unripened soils exposed to drying and atmospheric oxygen. Such mottling is evidence of improved aeration rather than of hydromorphism.

- A high content of organic matter or the presence of peaty layers in soils of humid climates can be regarded as an extreme example of moist soils. If iron oxide coatings on sand grains in podzol soils just below the B_2 horizon are absent, this can be regarded as a sure sign of present or past removal of iron in a reducing environment. After the removal of the iron, a fluctuating groundwater table can leave no tell-tale mottling or other colour features indicative of the height and depth of the water table. Mottling and black manganese concretions in a grey A_2 horizon and in the B_2 horizon of Alfisols (soils with high base saturation and marked illuvial clay accumulation in the B_2 horizon) are signs of a temporary perched water table in these horizons.

- Finally, an increase in soil salinity towards the surface in regions with arid and semi-arid climates is a sign of high water table.

In The Netherlands the depth to specific grey mottling (DE BAKKER, 1973) and the depth to gley horizon were correlated with actual groundwater regimes (SCHELLING, 1960). The behaviour of the water table was characterized in terms of the mean highest water table (MHW) and the mean lowest water table (MLW). It was found that in different areas and in soils from different parent materials the relationships between MHW or MLW and the depth to mottling or gleyed zones were similar but not identical.



LEGEND

Water-table class

	I	II	III ^a _b	IV	V ^a _b	VI ^a _b	VIIa	VIIb	
									a wetter variant of MHW
									b .. drier variant of MHW
									1) not occurring in the mapped area
MHW		< 40	< 40	40 - 80	80 - 120	> 120			
MLW	1)	1)	80 - 120	1)	> 120	> 120	> 200	> 200	MHW mean highest water table
									MLW mean lowest water table

Fig.1. Depth to water table classes distinguished on Dutch soil maps.

On Dutch soil maps seven watertable classes are distinguished, based on hydrological and hydropedological data (VAN HEESEN, 1970; see Fig.1). The duration of saturated conditions for various depths in the soil profile is not taken into account and neither are the frequency of low and high water tables or the periods in the year that they occur.

20.3.2 INFILTRATION RATE

In any drainage project it is essential to have a proper insight into the rate at which rain or irrigation water enters (infiltrates) into the soil (see also Chap. 5, Sect.4.2, Vol.I and Chap.15, Sect.4.1, Vol.II for different approaches to the phenomenon of infiltration). Water that does not infiltrate creates a nuisance on the land surface. It is of no use to the crops (excepting rice) and it does not help to leach any superfluous salts out of the rootzone. Too slow or too fast infiltration creates problems for the irrigationist. Hence infiltration tests are required, particularly in planned irrigation projects.

It can often be observed that the infiltration rate - also called intake rate - decreases with time. One can distinguish different physical quantities that characterize the whole or parts of the process of infiltration. These are:

- the *instantaneous infiltration rate* (I_{ins}), which is the volume of water infiltrating through a horizontal unit area of soil surface at any instant (infinitely small period of time). It shows, in general, a rapid decline in the beginning, followed by a more stable, very slow decline after some 3 to 4 hours of infiltration.

- the *cumulative infiltration* (I_{cum}), which is the total volume of water that has infiltrated through a unit of horizontal area of soil surface over a given period of time, measured from the beginning of infiltration.

- the *average infiltration rate* (I_{av}), which equals the cumulative infiltration divided by the time since infiltration started.

- the *basic infiltration rate* (I_{bas}), which is the relatively constant rate that develops after some 3 to 4 hours. A good criterion of the term "relatively constant" would be a change in infiltration rate of less than 10 per cent as compared with that of the preceding hour, or $(I_t - I_{t+1}) < 0.1 I_t$ (t expressed in hours). This is the physical quantity commonly used in hydrologic studies and runoff-erosion work.

- the *infiltration capacity* (I_{cap}) which is the maximum rate at which a given soil in a given condition can absorb water. When water is ponded over the soil during an infiltration test or an irrigation, one always obtains this maximum rate.

The (I_{ins})-time relationship is the basis for the design of sprinkler irrigation systems. In contrast, the (I_{cum})-time relationship is the basis for design of surface irrigation systems. The starting point in this reasoning is a crop's water requirement and a convenient irrigation schedule. However, for the purpose of this chapter it is necessary to know what portion of the water is removed "slowly" through the soil and what portion reaches a drainage ditch "rapidly" over the soil surface. The measured intake characteristics have to be interpreted in this context.

Field measurements on infiltration rate invariably yield data in terms of I_{cum} . Therefore it is much more convenient to derive an expression for I_{ins} by differentiating the expression for I_{cum} with respect to time, than to find I_{cum} by integrating I_{ins} . For many soils a plot of I_{cum} as a function of time is described by the equation

$$I_{cum} = at^n \quad (1)$$

where a and n are constants for a given soil and a given moisture content respectively. The constant n is positive but less than unity. From Eq.1 the instantaneous infiltration I_{ins} at any time t can be derived by taking the differential with respect to t

$$I_{ins} = \frac{dI_{cum}}{dt} = a n t^{n-1} \quad (2)$$

A convenient relationship is now

$$I_{ins} = \frac{I_{cum} n}{t} \quad (3)$$

From Eq.1 we can also obtain an expression for the average infiltration I_{av} by dividing I_{cum} through t

$$I_{av} = \frac{I_{cum}}{t} = a t^{n-1} \quad (4)$$

When Eqs.1,2, and 4 are plotted on double logarithmic paper, each one gives a straight line. Such plots are tantamount to rewriting these equations in loga-

rithmic form. Equation 1, for example, then reads

$$\log I_{cum} = \log a + n \log t \quad (5)$$

where a is the intersected part on the vertical axis, and n equals the tangent of the angle with the horizontal axis (Fig.2).

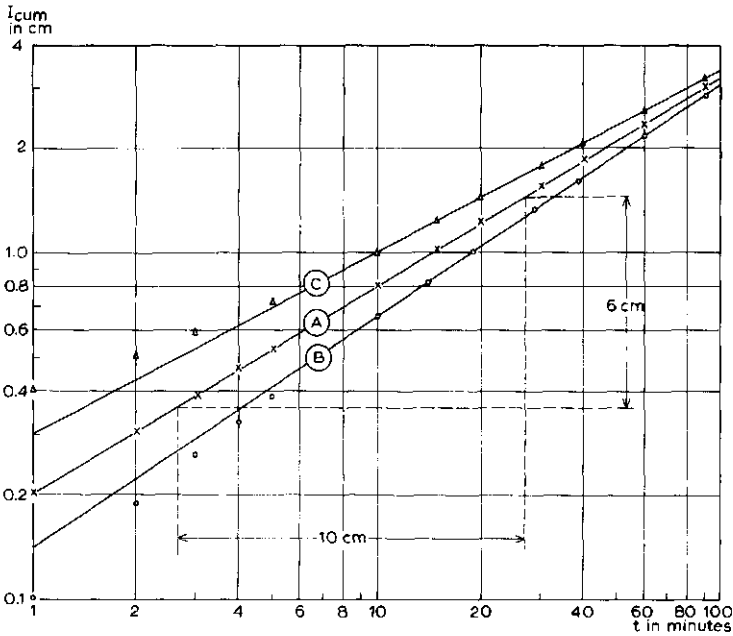


Fig.2. Plots of cumulative infiltration against time.

A: Correct measurements and no soil complications: $I_{cum} = 0.20 t^{0.60}$

as the intersect $a = 0.20$ and the tangent $n = \frac{6}{10} = 0.6$

B: Measurements are running 1 minute behind: $I_{cum} = 0.14 t^{0.67}$

C: Excessive infiltration during the first minute: $I_{cum} = 0.30 t^{0.52}$

The time that must elapse before the infiltration rate becomes approximately constant can be expressed in terms of the soil property, n . The criterion in the definition of the basic infiltration rate, I_{bas} , is used to set up the differential equation

$$\frac{dI_{ins}}{dt} = -0.1 I_{ins} \quad (6)$$

Differentiating the left-hand term one obtains

$$\frac{dI_{ins}}{dt} = \frac{d(a n t^{n-1})}{dt} = (n-1)a n t^{n-2} \quad (7)$$

The right-hand term of Eq.6 can also be written as

$$0.1 I_{ins} = 0.1 a n t^{n-1} \quad (8)$$

Substituting Eqs.7 and 8 into Eq.6 gives

$$t = 10(1-n) \quad (9)$$

For a description of infiltration tests, see Chap.24, Vol.III; BERTRAND (1965); HAISE, DONNAN and others (1956). Usually the data for three tests conducted within a small area are averaged to characterize a particular type of soil. The three test results are plotted separately on the same sheet of double logarithmic paper and, if the points for all three replications are close enough together, an average curve can be drawn.

In homogeneous soils deviations from a straight line can result from inadvertent delays in the first readings when the water has already started infiltrating and from the presence of small cracks and biogenic holes which are filled up very rapidly in the first minute or so. In Curve B (Fig.2) for example, the observer measures I_{cum} for $t = 1$ when in fact, $t = 2$. Comparison with Curve A shows that the real I_{cum} ($t = 2$) = 0.3 cm, which is 0.1 cm more than the real I_{cum} ($t = 1$). In this incorrect procedure the observer mistakes this increment of 0.1 cm for the real I_{cum} ($t = 1$).

In heterogeneous soils deviations from a straight line are to be expected. Soils having a strongly developed crack system and large biogenic holes show very high initial n -values (0.9 or more), which diminish after some time. In very sandy soils the n -value remains above 0.9 throughout the test. If the subsoil differs from the top soil in infiltration rate, the curve will show a change of slope when, after some time, the wetting front passes the boundary between top soil and subsoil.

In general, the n - and a -values are useful indicators of the infiltration characteristics of soils. The a -value in Eq.1 is a measure of the magnitude of infiltration and, for homogeneous soils, is independent of time. The physical meaning of the a -value can be illustrated by an example. If $I_{cum} = 0.2 t^n$ (I_{cum} in cm,

t in min.), it follows that for t = 1 minute, $I_{cum} = 0.2$ cm. For t = 1 hour, one reads the I_{cum} value at 60 minutes from the graph. The n-value on the other hand reflects the change in infiltration rate with time. In moist soils the a-values are lower than in dry soils and the n-values are higher.

When dealing with heavy, swelling clay soils the use of cylinder infiltrometers is likely to yield unrealistic results. If the area of soil enclosed by the cylinder comprises a large crack interconnected with a system of cracks outside the cylinder, the infiltration rate will be far too high. If no cracks are enclosed, the infiltration rate may be almost zero. Yet, the cracks are part of the reality for such soils; hence basin infiltration tests are indicated (ALLEN and BRAUD, 1966).

For classification purposes, representative infiltration data from the field can be grouped into distinct families of infiltration curves (S.C.S.National Engineering Handbook, 1964) and each family can be given a class name, such as rapid, moderately slow, etc. (see Fig.3).

A classification of basic infiltration rates is suggested by RICKARD and COSENS (1965) for irrigated soils in New Zealand:

Class	Intake designation	Basic infiltration rate (mm/hour)
0	very low	less than 2.5
I	low	2.5 - 15
II	medium	15 - 28
III	high	28 - 53
IV	very high	more than 53

VAN BEERS (private communication), stressing the convenience in irrigation of working with the total amount of time in hours required for the infiltration of 10 cm of water, has put forward a different tentative classification:

Class	Time required for 10 cm cumulative intake
marginal (too slow)	20 - 40 hours
somewhat unfavourable (slow)	8 - 20
favourable	1.5 - 8
somewhat unfavourable (rapid)	1.5 - 1.0
marginal (too rapid)	1.0 - 0.5

The basic infiltration rate is the most important characteristic for land drainage, and it should be considered in conjunction with the climate (Chap.18,Vol.III; Chap.33, Vol.IV).

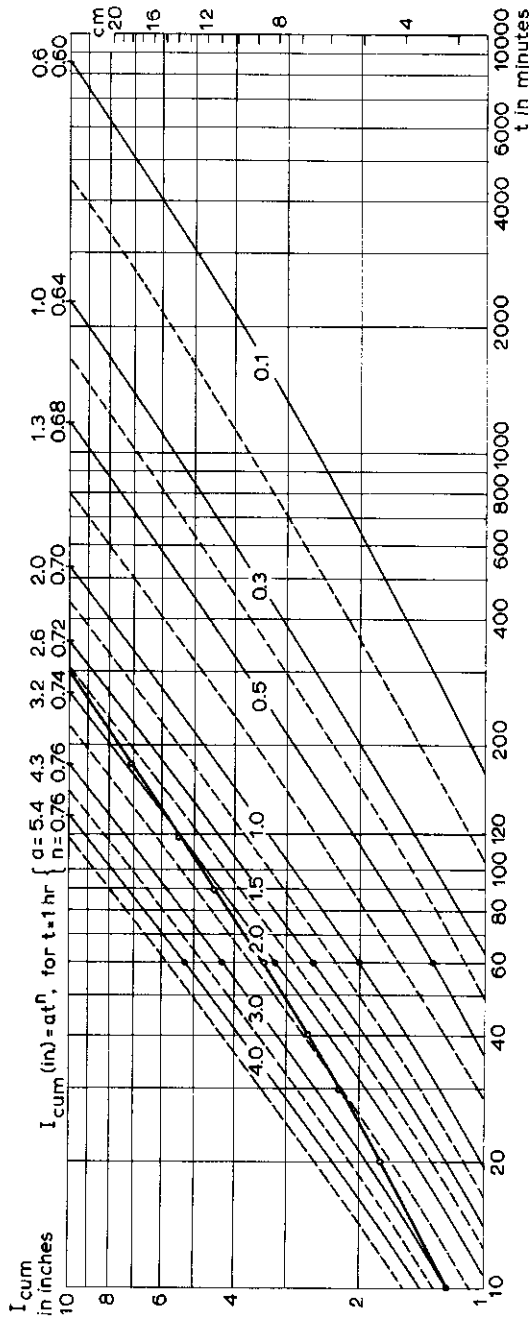


Fig. 3. Basic intake rate families for surface-irrigation design and a measured curve belonging to the 2.0th/hr family (I_{bas} occurs when $t = 10(1-n)$).

20.3.3 STRUCTURE STABILITY

Agriculturists and soil scientists are interested not only in the structural condition (Chap.2, Sect.3.2, Vol.I) of the soil as it affects intake characteristics during a hydropedological survey or even during a first infiltration test; they are also vitally interested in the deterioration or improvement of soil structure as a result of the action of water on the cohesive strength of soil aggregates, or as a result of future soil management when the area is brought under agricultural use. The collapse of soil aggregates when wetted with water is called slaking, and if the water surrounding the soil becomes turbid the soil is said to disperse. Upon drying, a slaked surface soil will often form a crust of low permeability, which may at the same time be so hard that germinating crop seeds cannot break through it.

If intake characteristics are to be considered favourable, the majority of soil aggregates, at least 3/4 by weight, must remain intact, i.e. must not become smaller than a predetermined size (diameter, say 0.25 mm), when subject to impacting raindrops or moving water. Low intake rates on sloping lands results in more run off, a higher attendant risk of erosion, and a need for more intensive surface drainage systems. Even salinization of soils can occur, as has happened for example in parts of the Jessore District of Bangladesh, where only 300 to 400 mm of the 2000 mm precipitation actually infiltrates into the fine, silty, poorly structured soils of the delta during the wet season. In the dry season, lateral capillary flow from the flooded depressions causes salt accumulation in a wide zone fringing these depressions.

Soil aggregate stability can be measured fairly simply in the laboratory. One somewhat aggressive technique employs wet sieving of air-dry aggregates, followed by a weighing of the aggregates remaining on the sieve after oven drying (KEMPER and KOCH, 1966). Another, more passive, technique is based on measuring the loss of pore volume due to slaking when air-dry aggregates are wetted with water and vibrated (WILLIAMS, 1963). Aggressive methods fail to show differences between soils whose aggregate stabilities are low to varying degrees. However, one may first place the soils in broad classes of aggregate stability by a technique which requires only a set of beakers, some distilled water, and a few other readily available items (EMERSON, 1967). Once one has a rough idea of the aggregate stability, one can then choose an appropriate method for making finer distinctions. The work by Kemper and Koch, which represents numerous soils in the western U.S. and Canada, indicates that

- aggregate stability declines very rapidly when soil organic matter contents fall below 2%, as is often the case in desert areas

- aggregate stability increases as soil clay contents are higher (about 75% stable aggregates at 40% clay)

- CaCO₃ content of the soil has no influence, and

- soils with exchangeable sodium percentages in excess of 20 have no aggregate stability at all.

For light sandy loams and silty loams in The Netherlands (BOEKEL, 1965) the risk of structure deterioration was correlated with the ratio between upper plastic limit (liquid limit) and moisture content at field capacity (pF = 2) as follows

$\frac{LL}{FC} > 1.10$ little or no slaking will occur

$\frac{LL}{FC} < 0.95$ risk of serious slaking during long wet periods

$\frac{LL}{FC} < 0.90$ risk of serious slaking during short wet periods

Between $\frac{LL}{FC} = 1.10$ and $\frac{LL}{FC} = 0.90$ the slaking hazards vary markedly with the manner and moment of tilling the soil in autumn.

20.3.4 PERCOLATION RATE

Percolation is not synonymous with infiltration. Infiltration refers to the entry of water at the soil surface; percolation refers to the quantity of water passing per unit of time through a horizontal unit area at a given depth within the soil mass. Only in a saturated soil does percolation equal infiltration; in unsaturated soil, infiltration equals the sum of percolation and storage.

Some authors, in defining percolation, prefer to think of the time required for a unit quantity of water to percolate through the soil. Unlike the concept of infiltration, it is not possible to define percolation in unambiguous physical terms that would suggest a simple field method of measuring percolation. Percolation tests reported in literature are highly empirical because the flow of water through walls and bottom of test holes takes place under undefined boundary conditions.

The test holes have to be auger holes or dug holes of known diameter and to a depth at which the percolation rate is to be measured, generally in the layer thought to have the lowest hydraulic conductivity. The test holes are then filled with water and one measures the volume of water that flows through the soil.

Several precautions must be taken to ensure the unhindered movement of water through the walls and bottom of the test holes; for example, the exposed surfaces of the hole should be sacrificed or a layer of gravel added to prevent scouring and sediment formation. Test holes are also pre-soaked to obtain the more representative percolation rate for wet soils.

To standardize the boundary conditions for the percolation test so that the measured results are quantitative, reproduceable, and physically interpretable, the test can be done as a normal infiltration test or with the inversed auger hole method (Chap.24, Vol.III).

With the infiltration method, one uses the normal cylinder infiltrometer, but it is now installed on the horizontal bottom of an excavation. The excavation should be made to a depth just above the layer that is suspected of being the least pervious part of the profile. The cylinder is pushed or hammered some 2 to 5 cm into this layer and the normal infiltration procedure is followed. The resulting measurement will yield information on the hydraulic conductivity of the limiting layer under nearly saturated conditions (stabilized moisture content).

In land drainage the inversed auger hole method is often applied for layers above the water table. The hole must be augered to the depth of interest, usually into, but not through, the layer of limiting hydraulic conductivity. In experimental fields with subsurface drains and ditches, the ratio of "rapid" to "slow" ditch discharge, following rain or irrigation, reflects the quantities and rates of surface flow and percolation.

The macropores in a soil profile play an important role as conduits for water, and hence largely determine the percolation rate. Pores in a soil profile originate from (decomposed) plant roots and microfauna (ants, worms, molluscs). A single tubular soil pore (tortuosity = 2, diameter = 1 mm) per m² horizontal cross-section can give a hydraulic conductivity of 0.01 m/day. In various studies, counts of up to 1,500 of the larger, easily visible pores have been reported. Pore counts have successfully been correlated with reclaimability of saline-alkali soils in West-Pakistan (ALIM et al., 1971). During a hydropedological survey due attention should be given to pores visible in profile pits or in undisturbed samples deeper down. An experienced hydropedologist may even be able to estimate the hydraulic conductivity so that field measurements can be economized on.

Like pores, cracks play a major role as conduits for water. In soils that are commonly cracked, determining the hydraulic conductivity on small core samples gives unrealistic results (5 to 25 times too low). In some swelling soils the cracks may close entirely upon saturation, but in others the cracks remain partially open. There are reports of clay soils that are permanently below the water table and yet have wide-open cracks (KALLSTENIUS, 1963; DIELEMAN and DE RIDDER, 1963).

The phenomenon that the cracking of drained and drying soils in the IJsselmeer polders (The Netherlands) has contributed greatly to the spectacular increase of hydraulic conductivity is an accepted fact.

The effect of cracks and soil structure on hydraulic conductivity can be calculated, as has been shown by BOUMA and HOLE (1971) and ANDERSON and BOUMA (1973), but their approach has not yet been applied on a practical scale.

20.3.5 HYDRAULIC CONDUCTIVITY, TRANSMISSIVITY

The hydraulic conductivity of the soil profile at and below drain depth is the most important soil physical characteristic to be determined in any subsurface drainage project. The magnitude of the hydraulic conductivity of a soil depends on pore geometry and the nature of the particle surfaces. Thus it varies with texture, structure, density of packing of soil grains, and grade of cementation. The effects of texture, structure, and density are in turn modified by the content of organic matter of the soil, the degree of saturation with Ca or Na, and the type of clay minerals (swelling or non-swelling). Some of the factors controlling hydraulic conductivity can vary considerably over short distances, or at a given site over short periods of time.

For sandy sediments which have a clay plus silt content of less than 6 per cent, quick and reliable estimates of hydraulic conductivity can be made on the basis of particle size distribution or on the specific surface U of the particles (Chap. 24, Vol. III). For the intermediate and heavy textured soils, other factors not related to particle size generally far outweigh the influences of particle size. Attempts have been made (Table 2.2 from ZIMMERMAN, 1966; HORN, 1971) to consider these other factors in arriving more cheaply at usable field estimates of hydraulic conductivity. The merit of this work is not to be found in any universally applicable relationships but in pointing out the possibility of developing a fairly consistent local relationship between hydraulic conductivity and easily observable and measurable soil properties.

HORN's (1971) curves (Fig.4) represent the approximate extreme limits of hydraulic conductivity for natural soils. A highly sorted sand with spherical grains will have a hydraulic conductivity that is somewhere on or near the theoretical curve, depending on the particle size. A clay loam, in contrast, could have a K-value almost anywhere on a horizontal line extending from the point representing its mean particle diameter. Thus a clay loam with high ESP and swelling clay minerals may have a K of 0.01 m/day, while one with a good and stable structure may have a K of 8 m/day.

The common opinion that clays and clay materials always have low hydraulic conductivities is unjustified. In different parts of the world there are heavy clays whose hydraulic conductivity in natural condition is very high and comparable with that of coarse sands and gravel. DIELEMAN and DE RIDDER (1963), in studying the hydraulic conductivity of a 2 m thick heavy clay layer occurring along the borders of Lake Chad (Central Africa) found values as high as 1000 to 5000 m per day, the reason being wide cracks and fissures, which had not closed after the clay had been inundated by the lake water in the geologic past.

Another example is that of the heavy humic clays which occur locally in the Danube flood plain in Roumania. For these structured clays K values as high as 30 m per day are reported (FAO Work Document AGL:UNDP/ROM.1).

It is the hydropedologist's major task to determine the magnitude of the hydraulic conductivity at and below drain depth and to indicate how it varies within the boundaries of the project area. Although various methods for measuring hydraulic conductivity exist, the auger hole method is the one most commonly applied (Chap. 24, Vol. III).

To calculate the spacing of drains, the value of the hydraulic conductivity, K, is by itself insufficient. Also required is information on the thickness, D, of the water transmitting layers and on the depth of the impervious base layer. If a continuous layer of heavy clay is found at some depth in a light textured soil profile, such a clay layer may be regarded as the impervious base of the profile. But an impervious layer need not be "impervious" in the true sense of the word, i.e. a layer which does not allow any passage of water. In drainage, "impervious" has a relative meaning. A base layer whose hydraulic conductivity is 10 times less than that of the overlying material may be considered impervious in drainage.

In a homogeneous soil, the depth of soil through which the major part of the groundwater flows towards the drains equals roughly 1/4 of the drain spacing. Hence it is obvious that a conventional soil survey, which provides information on only

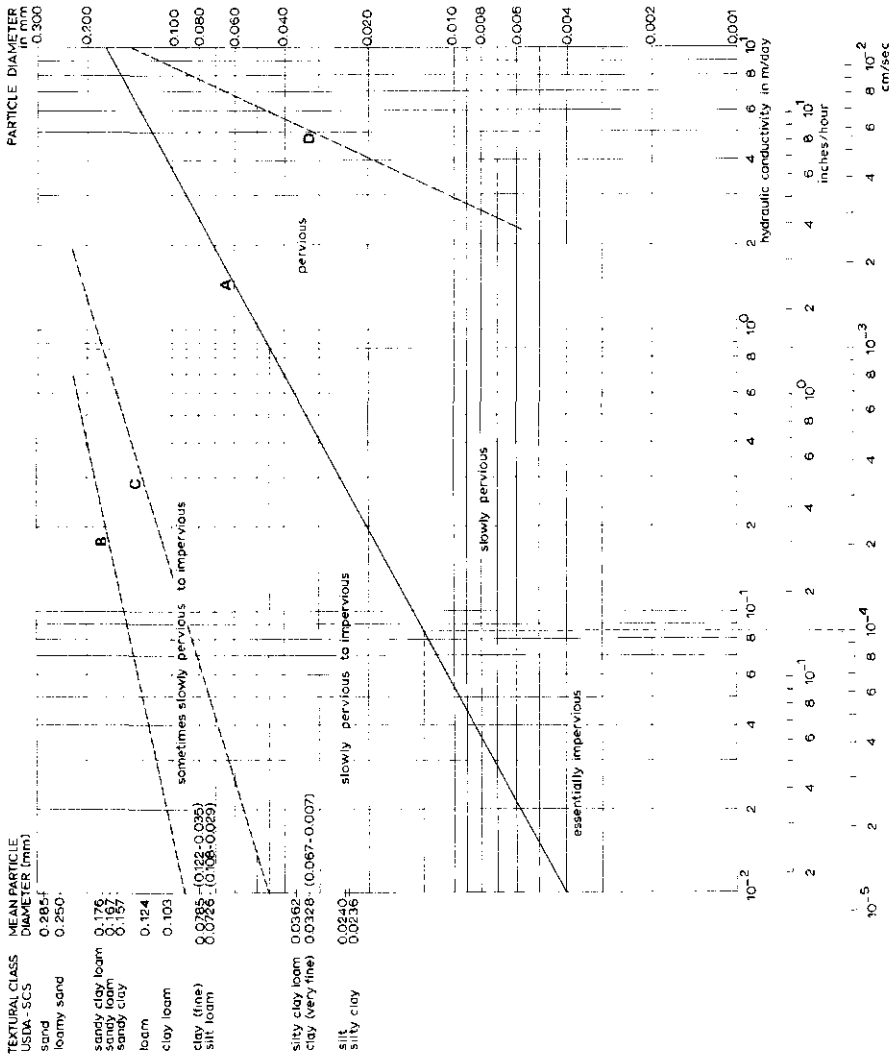


Fig. 4. Hydraulic conductivity in relation to mean particle diameter and soil structural conditions (HORN, 1971).
 A: Theoretical permeability as function of particle size. B: Soils with high exchangeable sodium percentage, highly dispersed swelling clays. C: Pydled soils of poor structure, highly compacted. D: Soils with good structure, highly flocculated due to abundant Ca, organic matter, iron oxides, mechanically loosened, noncompacted non-swelling clays.

the upper 1.20 m of the soil profile, will not suffice. With a soil auger and extensions, however, the hydropedologist should be able to reach a depth of 5 to 6 m, thus obtaining the information on soils and soil hydrological properties required for a drain spacing of up to roughly 25 m.

For expected drain spacings of more than 25 m, it will be necessary to explore the subsurface geology by deeper borings, say to 20 m. To determine the natural drainage of the project area or the inflow of groundwater from adjacent areas, it may be necessary to make even deeper borings, say to 50 or 100 m, depending on the subsurface geological conditions.

Here is perhaps the right place to remark that a hydropedological survey differs from a normal soil survey, not only in its depth range, but also to some extent in its approach. A soil surveyor usually makes ample use of air photographs and all existing knowledge of the physiography and surface geology of the terrain, which allow specific land features, physiographic units, and major soil groups to be delineated. Each unit or group is then explored by means of bore holes, soil pits, and road cuts, and the resulting profile descriptions are used to compile a soil map.

Standard soil augers without extensions generally do not reach deeper than 1.20 m and the depth of a pit is often of the same order. The geology of the deeper, unexplored layers is usually not reflected in the nature of surface features. For example, river channels and natural drainage patterns in alluvial plains and valleys have altered repeatedly in the past. As a result of these changing depositional environments, we may expect a large variation in the vertical and horizontal distribution of sand and clays and other materials of the underground. Thus, even though a conventional soil map provides a good basis for selecting appropriate sites to measure infiltration rate, percolation rate, etc. it cannot serve as a guide for the selection of sites for the vital hydraulic conductivity and transmissivity measurements at and below the anticipated drain depth. Therefore, when such measurements are to be taken, it is common to do so on the basis of a grid system, this being the most objective method. The grid can be a system of squares or a number of approximately parallel rows, the distance between the rows being greater than the distance between the observation sites in the rows. Examples of grid systems and criteria for their lay-out are given in Chap.36, Vol.IV. Observations of the soil profile, the subsurface geology, the water table, and the groundwater quality are made at the nodal points of the grid. Hydraulic conductivity should be measured at regular depth intervals, special attention being given to the layers at and below the anticipated drain depth.

If details of the subsurface geology of the project area are known from previous work (geological survey, drainage reconnaissance survey) it is sometimes possible to delineate a number of sub-areas which can be regarded as homogeneous hydrogeological units. Such a unit may be one in which the impervious layer is found at about the same depth, or one in which the texture of the water-transmitting zone, although varying, still varies less within the unit than outside it. Within these homogeneous hydrogeological units the number of hydraulic conductivity measurements can be reduced, i.e. it will not be necessary to take measurements at each nodal point of the grid. Similarly, other observations can be economized on, for example, those of the chemical composition of the groundwater.

The density of the grid system will depend on the required accuracy of the survey and on the anticipated drain spacing. The closer the drain spacing, the narrower the network will be. One boring per hectare means a very intensive survey and will usually only be required in small complicated areas. For reconnaissance surveys one boring per 4 to 10 hectares and for broad reconnaissance surveys one boring per 25 to 100 hectares are fair averages. These densities are given only as a guide and it is by no means suggested that they be rigorously applied everywhere in the world.

Once the hydraulic conductivity values are known, they are plotted on a map. If the values for each depth interval do not differ greatly, a weighted mean K-value can be calculated for each nodal point, and lines of equal hydraulic conductivity can be drawn (Fig.5).

If the K-values differ markedly over the measured depth range, as happens in layered soils, no mean is calculated but instead the initial values are plotted on the map.

It should be remembered that minor or even relatively great variations in hydraulic conductivity do not greatly affect drain spacings because drain spacing varies with the square root of hydraulic conductivity. Hence, given a certain thickness, D, the value of the hydraulic conductivity, K, may vary quite a lot without much affecting the drain spacing.

One thing that should be kept in mind is that too many observations and measurements are a burden to everyone; they do not necessarily give a clearer picture of the situation and they certainly make the venture very costly. It depends on the skill and experience of the hydrogeologist whether he is able to collect the maximum amount of information with a minimum of costs.

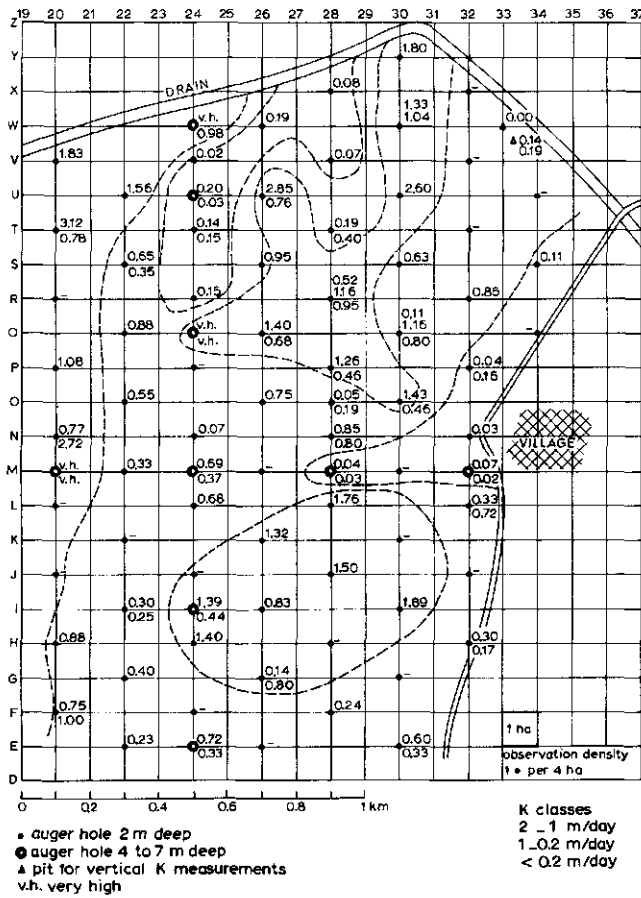


Fig.5. Hydraulic conductivity map of part of a projected pilot area. Where more than one figure is shown the upper value refers to a test at approximately 1 m depth. Otherwise the K-values refer to depths of between 1.5 and 2.0 m.

The results of the 5 m deep borings should also make it possible to prepare

- a contour map of the impervious base (depth contours)
- a map showing any impeding layers
- a map showing any coarse sandy and gravelly layers at or below expected drain depth.

20.3.6 SOIL MOISTURE STORAGE CAPACITY

The storage capacity, i.e. the quantity of moisture that can be stored in the soil, depends on the pore volume and the depth at which the groundwater table is found.

The storage coefficient μ - also called effective porosity or drainable pore space - is a measure of the change in water table under given conditions of recharge or discharge. Hence, it is a soil characteristic that must be determined in any subsurface drainage project. Two definitions of storage coefficient are in use, depending on the depth at which the groundwater table occurs:

$$1) \mu = \frac{\text{change in groundwater storage } (\Delta S_{grw})}{\text{change in groundwater level } (\Delta h)}$$

This applies to soils in which the groundwater occurs at such a depth that fluctuations in the moisture content of the upper layers are transmitted to the groundwater only after long delays and with greatly reduced amplitude (Chap.22, Vol.III).

$$2) \mu = \frac{\text{change in storage capacity}}{\text{change in groundwater level}} = \frac{\text{change in moisture content } (\Delta S_m)}{\text{change in groundwater level } (\Delta h)}$$

This applies to soils in which the groundwater occurs at relatively shallow depth, so that soil moisture fluctuations in the soil profile or the root zone of crops are closely reflected in water table changes.

Figure 6 shows the equilibrium moisture distribution above varying groundwater levels for two homogeneous soils, a sand and a clay. Also shown are the pF curves.

Let us consider the situation where the water table is at a depth of 100 cm. An equilibrium moisture condition curve DEF can be constructed. If there is equilibrium in the soil profile, the suction of the soil moisture at any point above the water table equals the height of that point above the water table. Hence the suction at the surface is 100 cm, corresponding to pF = 2. Projecting F' into the moisture content axis yields the associated moisture content, F. At and below

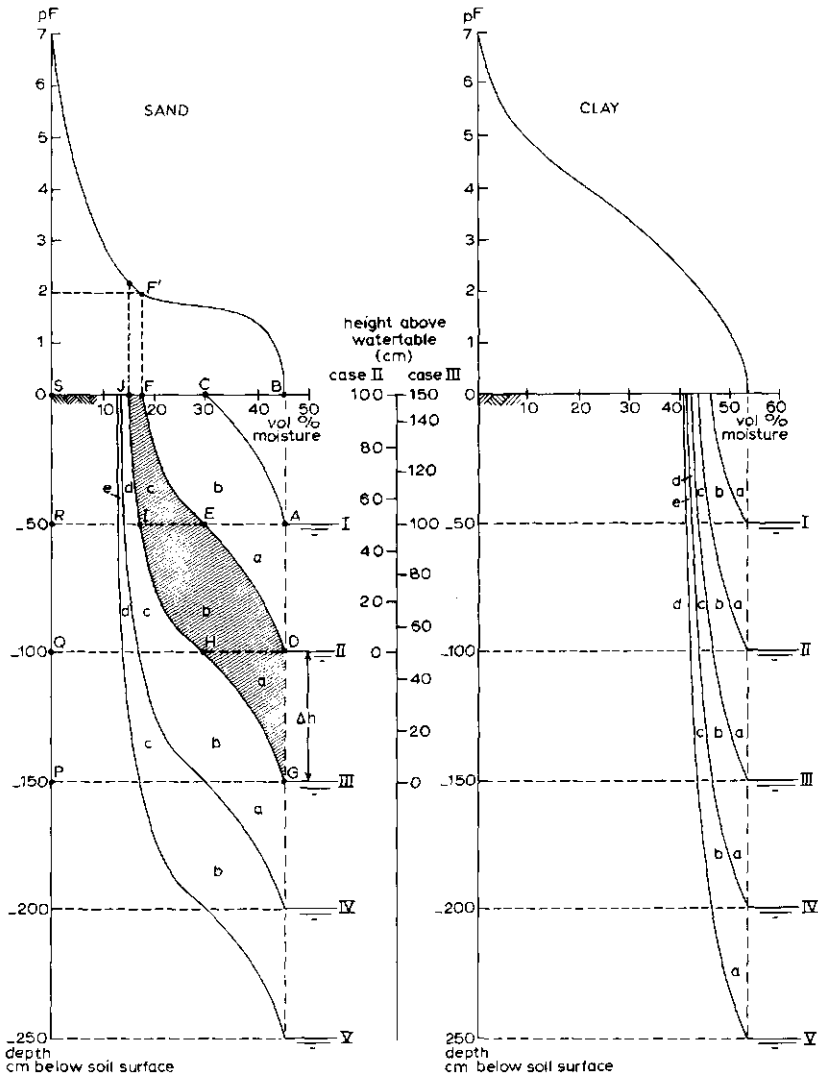


Fig.6. Change in moisture content and storage capacity as a result of changing water table levels.

the water table the sand is fully saturated with 45 per cent moisture content (point D). In this manner we can find all intermediate moisture contents, which results in the curve DEF. The area DFSQ represents the total amount of moisture in the 100 cm layer above the water table, while the area DBF represents the maximum possible storage capacity if the water table were to rise to the surface.

Let us now consider the situation where the water table has dropped over a distance Δh to a depth of 150 cm. A new equilibrium moisture condition curve GJ is now established, whereby part GI of the curve is identical to curve DF, and consequently area DBF is identical to the area of GAI. The maximum possible storage is then represented by the area GBJ.

The areas representing storages in these two situations can be divided into three sub-areas, a, b, and c, which are always congruent when the soil is homogeneous in a vertical direction. By lowering the water table from 100 to 150 cm, the storage capacity is increased by the area GDFJ, which is equal to (a+b+c) and to the area BJIA.

As the area BJIA is approximately a trapezium, one can write the second definition of the storage coefficient as

$$\mu = \frac{\text{area BJIA}}{\Delta h} \approx \frac{BJ + IA}{2} = \frac{BJ + BF}{2}$$

This means that μ is equal to the mean soil air content at the soil surface between the two equilibrium states (II and III). When the water table is deep, the area BJIA approximates a rectangle, so that for a water table change of Δh

$$\mu = \frac{BJ \times BA}{\Delta h} = \frac{BJ \times \Delta h}{\Delta h} = BJ$$

or μ equals the soil air content at the surface. It is clear that μ is not constant but increases with the depth of the groundwater table. However, for water table changes at depths below 150 cm, μ may be taken as a constant value.

Peculiar effects can occur when nearly full capillary soil pores reach to the soil surface and water tables are at very shallow depths. In such a situation μ will be almost zero and a small excess of precipitation will cause the water table to rise to the soil surface or very nearly so. Similarly, a small net evaporation of just a few millimetres may cause the water table to drop several tens of centimetres.

As was shown in Table 1, determining the storage capacities of soils is part of a soil drainage study, usually concerned with the layers between 0.30 and 1.20 m below soil surface. This means that a conventional soil map can indicate the proper sites to collect undisturbed core samples. Methods of determining typical or average pF curves of samples are described in Chap.23,Vol.III. The drainable pore space μ can also be measured in a pilot field, but this is only possible if the water table is not too deep and if all the terms of the water balance are known quantities (Chaps.22 and 26,Vol.III).

20.3.7 SOIL MOISTURE RETENTION

For soils which show stratification, as many alluvial soils do, there is only one practical way to obtain a reliable picture of the available water content of the rootzone; this is at the same time the least sophisticated way. Briefly the method is as follows:

Close off with planks an area of about 1 m² or use a cylindrical section of a large oil drum. Apply about 15 cm of water (the quantity of water can be adjusted in line with the desired depth of investigation; dry soil with 50 per cent moisture retention will be wetted 30 cm deep by 15 cm of water). Note the time required for infiltration. Cover the surface with plastic film or any other material such as straw to prevent evaporation. Return after two days, early in the morning if the days are hot and dry, and take samples at several depths, depending on the stratification or soil horizons of the profile. Weigh the filled containers soon after sampling and have the oven-dry weight and the wilting point determined in the laboratory (Chap.23,Vol.III). If one wishes to express the soil moisture content on a volumetric basis, the bulk density of the core samples must first be determined (Chap.2,Vol.I).

As a general guideline for irrigation projects, VAN BEERS (1972) suggested the following rating of total available water capacity for the rootzone:

Class	Total available water capacity in mm
favourable	more than 150
moderate	150 - 100
marginal	not less than 90 mm in the rootzone
lower limit	not less than 50 mm in the upper 60 cm of depth

The quantity of moisture available in the soil is closely related to soil composition and structure. SALTER and WILLIAMS (1965; 1967), SALTER et al.(1966), PETERSEN et al.(1968), PIDGEON (1972) and others have all established relationships

between these properties and available moisture. Generally speaking, field capacity moisture content depends on both structure (porosity, pore geometry) and texture, while at wilting point the water content depends more exclusively on the total surface area of particles, hence on clay content.

A comparison between relationships developed for various soils in different parts of the world is only possible if the same variables and identical methods of measurements have been used. These conditions are rarely satisfied. With this in mind, Fig.7 may serve as an example of soil texture related to available soil moisture.

To enable British agricultural advisors to make quick estimates of available water, SALTER and WILLIAMS (1967) devised a modified triangular texture diagram on which estimated values of available water capacity are superimposed as contours (Fig.8). To do so, they have to make a field estimate of % coarse sand and % silt plus clay, or they can resort to existing soil texture data from the laboratory. This diagram cannot be used by those working with the conventional USDA textural classes.

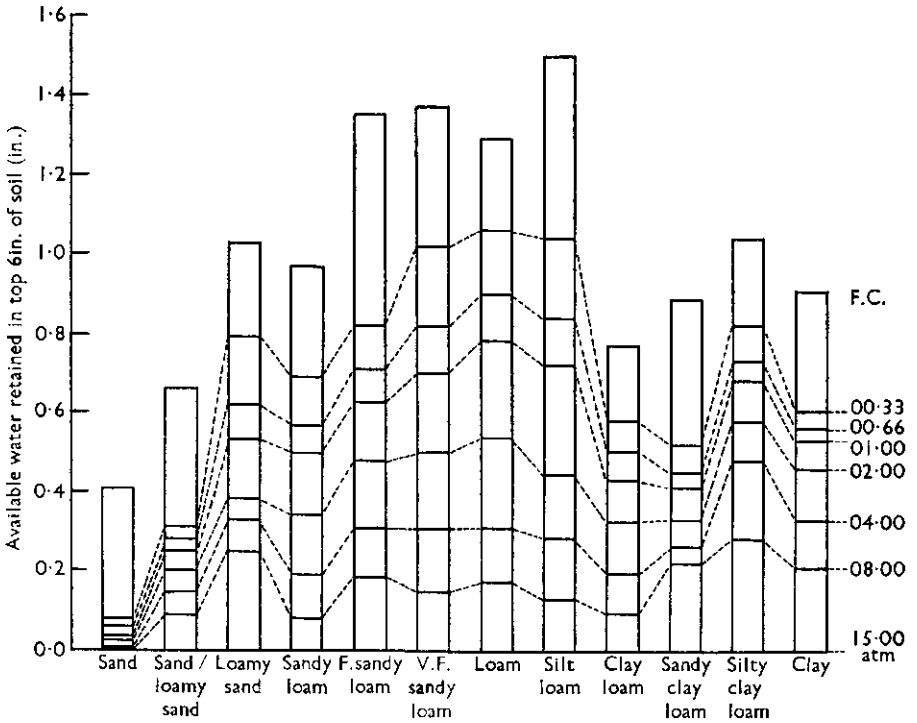


Fig.7. Water retention characteristics summarized for the different textural classes. (Salter and Williams, 1965)

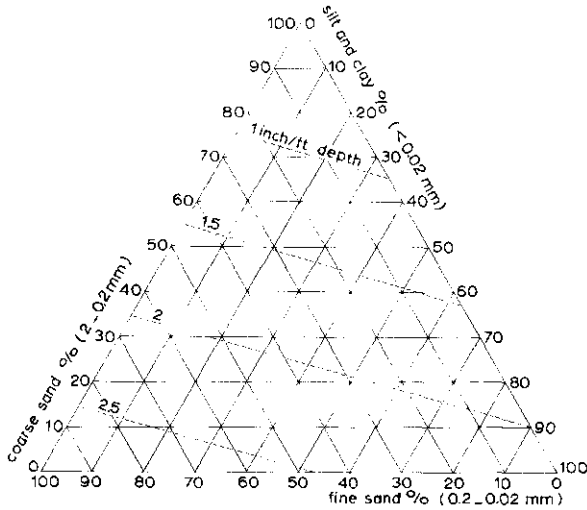


Fig.8. Modified triangular co-ordinate diagram. The estimated values of AWC (available water capacity) are superimposed as contours (broken lines) at intervals of 0.5 in./ft. depth. Contours are based on the regression equation: $AWC (in./ft.) = 2.17 - 0.018 (\% \text{ coarse sand}) + 0.0072 (\% \text{ fine sand})$. 1 inch/foot = 8.33 vol.%. (Salter and Williams, 1967).

20.3.8 SALINE AND ALKALI SOILS

Salinization of soils may be caused in different ways and under different climatological conditions (Chap.3, Vol.1), but the main cause is usually poor drainage. In arid and semi-arid regions salinity symptoms may not always be due to present poor drainage but could be a result of poor drainage in the geological past. If saline or alkali soils are suspected, the hydropedologist should give due attention to

- degree of present soil salinity, if any. This is measured in terms of EC_e values for each of the various layers (USDA Handbook 60, 1954)

- severity of present alkali conditions, if any. The concentrations of Na, Ca, and Mg in the saturation extract are measured for each of the various layers, and the SAR-values calculated. ESP-values can be determined or estimated from the SAR-values (USDA Handbook 60, 1954)

- all available field clues as to the source of the salts, the local processes of salinization, and whether these processes are still active and possibly intensifying, or whether one is dealing with fossil phenomena.

Good field indicators of soil salinity in arid regions are the presence of salt crusts and halophytic vegetation. An almost total absence of vegetation may point in the same direction. If a soil profile in an arid region is observed to be completely moist while there are no apparent reasons for such moistness, e.g. a previous rainfall or an irrigation, the explanation may lie in the hygroscopic action of high salt concentrations. The distribution of salt throughout the profile, as observed from accumulations of salt or gypsum crystals, or measured in the laboratory, may disclose the mode of salinization and the environmental factors influencing it. Figure 9 shows some schematic examples of salt distribution in a soil profile. Diagram A is a salt-build up typical for external solonchaks whose upper soil layers are in contact with saline groundwater (capillary rise). Diagram B shows the salinity distribution of an internal solonchak whose upper soil layers have no contact with the groundwater. Diagram C shows a homogeneous distribution of salt throughout the profile, which is a typical result of irrigation with saline water and a water table too deep to influence the profile's salt content.

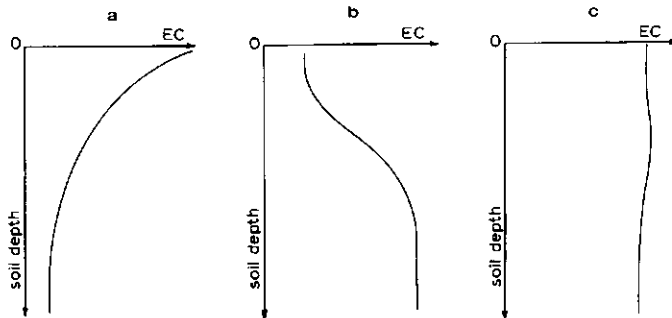


Fig.9. Schematic examples of salt distribution in soil profiles.

The hydrogeologist should assess salinization and alkalinization hazards in the future state when irrigation and drainage conditions will be different from those of the present state. In general, the introduction of irrigated agriculture will cause the water table to rise and this may create salt and alkali problems. Hence the hydrological properties of both shallow and deep soil layers and the groundwater characteristics (EC, SAR) must be investigated, thus allowing the hazards of a rapidly rising water table to be predicted.

Soils having silty or loamy textures at the surface and changing gradually into more sandy material at depth offer particularly favourable conditions for a rapid upward capillary flow of water from shallow or moderately deep groundwater. In

arid climates such soils easily develop salinity problems. If there are significant areas of such soils within a project area they should be mapped.

The gypsum content of the rootzone should be determined because this mineral yields the Ca which is essential for the reclamation of alkali soils. The soils in arid climates generally contain gypsum but the hydropedologist should find out whether there is enough of it in the rootzone to replace a harmful excess of Na. In semi-arid to sub-humid climates the soils rarely contain sufficient gypsum, so that special applications are required to prevent the soil structure from deteriorating.

For a similar reason it is useful to determine the calcium carbonate content of a soil, particularly if the soil contains no gypsum.

Saline clays containing much montmorillonite are difficult to leach, so it is a good idea to investigate the clay mineral composition of soils. The salt content and the Ca/Na ratio of irrigation water used for the leaching of soluble salts must be known as these qualities affect the drainage criteria. Water that is highly mineralized has to be applied in greater quantities, so more water will have to be discharged by the drainage system (Chap.9,Vol.II).

Concerning salinization, it is worthwhile to mention that a given regional or local climate is not identical with the "soil climate" in the same area. In soils with surface sealing, hardly any water will percolate, but there will be a great deal of surface runoff; even if rainfall is adequate, capillary rise from shallow groundwater will cause (re-)salinization of the topsoil.

20.3.9 LAND SUBSIDENCE AND SOIL RIPENING

Drainage of peat soils and muddy sediments poses specific problems, which are discussed in detail in Chap.32,Vol.IV. When such soils are drained, this is accompanied by substantial water losses, an increase in effective stresses, and the oxidation of organic matter by biochemical action. This results in the land surface subsiding, which affects the functioning, economics, and hence the design of the entire drainage system. Irreversible drying of the top layer of peat soils may form an accessory agricultural problem which also affects the drainage design because, to prevent it or to cure it, very close control of the water table is required. If sulphides are present in excess of calcium, their oxidation results in extreme soil acidity.

On the credit side, when the drainage of peats and muddy sediments is properly carried out, it will be accompanied by favourable "ripening" processes, such as an improvement of the soil structure, a great increase in hydraulic conductivity (due to formation of cracks), an exchange of Na^+ and Mn^{++} for Ca^{++} in tidal deposits, and the colonization of the drained soils by soil-inhabiting organisms. Drained muddy sediments in The Netherlands give yields well above average. Drained organic soils in many countries are used for intensive high-value vegetable production.

In drained peat soils, oxidation and the resulting slow subsidence are continuous processes; thus the thicker the peat deposits the longer they can be farmed. If the underlying mineral subsoil has favourable agricultural qualities, agriculture can continue after the peat soil has disappeared. If the mineral subsoil has no agricultural potential, the cost of the drainage scheme will have to be written off over the time that the organic soil can be farmed.

Empirical relationships have been established in various countries to predict total subsidence. Each method employs several of the following factors:

- total thickness of the compressible layer
- initial moisture content (weight percentage)
- organic matter content (weight percentage)
- clay content (weight percentage)
- intended depth of drainage
- initial specific volume and probable final specific volume (specific volume = inverse of bulk density)
- consolidation constants of the compressible materials
- initial and predicted final effective stresses (effective overburden pressures).

The laboratory tests on consolidation and the calculations of the rate and degree of subsidence are the work of civil engineers. The hydrogeologist should, however, map the kind and thickness of peats and other soft soil materials. He can take representative samples, disturbed or undisturbed, as the various laboratory tests require. He should also prepare contour maps of the relief of the underlying, less compressible mineral soils, which may necessitate deep borings. For peat soils with a relatively short lifetime after drainage, he should assess the agricultural potential of the mineral subsoil.

The future rootzone of muddy unripened sediments should also be tested for the presence of excess sulphides, which may cause the formation of acid sulphate soils after drainage (Chap.32, Vol.IV).

20.4 FINAL REMARKS

In this chapter emphasis has been laid on the physical and hydrological properties of soils which have to be determined by a pedologist working in a drainage project. Depending on the kind of drainage problem to be solved, the qualitative and quantitative data required will differ, though certain items are common to all drainage projects.

One such item is the conventional soil map, which gives information on the upper 1.20 to 1.50 of the soil profile. This map will serve as a basis for determining the agricultural potential of the project and will enable a study of the problems of soil drainage and surface drainage. In subsurface drainage too, the conventional soil map will provide valuable information on such matters as soil layers, the occurrence of impeding horizons, hardpans, etc., in the upper 1.20 or 1.50 m of the profile.

But what exactly do we mean by a "conventional" soil map? Soils are mapped by many different standards and procedures, and soil maps may be based on:

- soil genesis (processes which have played a role in soil formation)
- morphometry (observable and measurable soil properties)
- physiography (delineated on the basis of distinct land forms)
- land systems (delineated on the basis of complex landscape elements which may belong to more than one land form)
- single value (magnitude of a single parameter)
- a series of single values (combination of more than one parameter)
- direct subjective estimation of soil or land qualities (suitability maps).

A soil map prepared on the basis of physiography or land systems will be of little or no use for design purposes as it only portrays associations of sometimes quite diverse soils and does not show the location and extent of all distinct kinds of soils. Similarly, a map showing areas where certain soil-forming processes have been in play is of little value to the design engineer.

The soil map most appropriate for design purposes is the one prepared on the basis of soil morphometry. To describe the way in which such a map is prepared would be to go beyond the scope of this chapter, so suffice it to say that ample use is made of aerial photographs, geological and physiographical data, soil pits, hand borings, etc. Such a map can be of great use in studying soil drainage problems, but it does not provide the data required for designing a subsurface drainage system. Hence, if the problem to be solved is one of subsurface drainage, and even if one has a "conventional" soil map at one's disposal, a hydropedological survey will still be needed.

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SURVEYS AND INVESTIGATIONS

21. GROUNDWATER SURVEY

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PURPOSE AND SCOPE

A brief account of the basic elements of a groundwater survey related to land drainage.

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21.1 COLLECTION OF GROUNDWATER DATA

A characteristic feature of a groundwater system is that it reacts to various factors of recharge and discharge in a deterministic or a probabilistic manner. A deterministic system is one that is defined by definite cause-and-effect relations. Consider, for example, the abstract nature of an irrigation application and its distribution (a cause). This can be related to the change in water table (an effect). For a given quantity of irrigation water supplied, the change in water table can be measured. It will be clear that this cause-and-effect relation will remain abstract until it is related to the soil and geological conditions of the system. The function of a deterministic system is that of predicting effects arising from causes.

A probabilistic system allows no precise prediction. It may, however, provide expected values within the limitations of the probability terms which define its behaviour. A stochastic, or random, variable is a variable quantity with a definite range of values, each one of which, depending on chance, can be attained with a definite probability. If any one aspect of an otherwise deterministic relation is random in nature, the whole relation is rendered stochastic. An example is the continuity equation of a groundwater system

$$\text{inflow} - \text{outflow} = \text{change in storage.}$$

Since precipitation is usually random, replenishment of the groundwater by precipitation is random with an associated probability distribution. This renders the above equation stochastic, regardless of its deterministic elements. In practice, however, groundwater systems are usually treated in a deterministic manner.

An essential part of a drainage investigation is a survey of groundwater conditions. But it will be immediately understood that such a survey is not merely a matter of measuring depth to water table. We are, of course, interested in the existing state, in the extent and degree of the drainage problem; we wish to know exactly where high water tables occur and when. But a survey of only this element of the groundwater system is liable to end in failure because it cannot be treated separately from other elements with which it is closely interconnected. To solve a drainage problem effectively and efficiently we also need to know the actual cause of the problem.

To find it, we must analyze and evaluate the various factors that recharge and discharge the groundwater system and determine their effect on the water table. The best way to do this is by assessing a groundwater balance.

Thus the main objectives of a survey of groundwater conditions that forms part of a drainage survey can be defined as:

- determining the extent, degree, and nature of existing or potential drainage problems
- analyzing the groundwater system and assessing a water balance from which the cause(s) of the drainage problem may be understood
- indicating how the groundwater system can be altered artificially so that the resulting water table will not hamper crop growth.

This chapter is devoted to the collection, processing, and evaluation of groundwater data. The subject of the assessment of groundwater balances will be treated separately in Chapter 22, Vol.III.

21.1.1 PREPARATORY STUDY

The first step in a survey of groundwater conditions must always be a preparatory phase. This involves:

- obtaining suitable topographical maps on as large a scale as possible: 1/10 000, 1/20 000, or 1/50 000. These maps should show contours of the land surface. They are the essential foundation without which no accurate work can be done. If the area under study is a large one it is also convenient to have a topographical map on a reduced scale, say 1/100 000. Specific information can be transferred to such maps, which greatly facilitates studies in the office. They are also useful for reporting because there is nothing more annoying than bunches of maps folded ten times or more;

- obtaining air photographs which can help to make up for any shortcomings in topographical and geological maps. These photographs may provide useful information not only on topography and geology but also on hydrogeology and vegetation cover;

- obtaining geological maps, showing not only the outcrops of all the geological formations but also their lithology. These maps and geological cross-sections allow permeable and less permeable zones to be distinguished, zones of recharge, transmission, and discharge to be delineated, and the types of aquifers to be defined (unconfined, semiconfined, Fig.1). From this information the type of landscape can be determined and thus, in general, the groundwater conditions that can be expected (see Chapter 1, Vol.I).

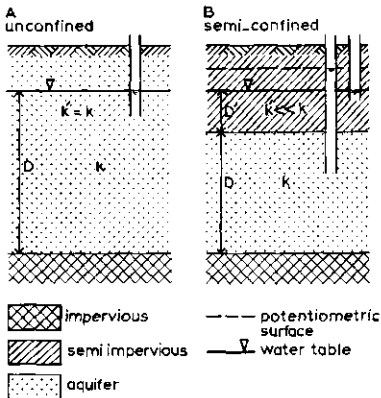


Fig.1. Schematic sections through an unconfined aquifer (A) and a semi-confined aquifer (B)

- obtaining hydrologic data from offices and departments. Since surface water and groundwater are interrelated, it is essential that all available data on surface waters be collected, including information on precipitation, river flows, springs, lakes, irrigation water, etc.;

- searching for water quality data. In many cases drainage problems are associated with salinity problems. Hence it is essential to have an idea of the groundwater salinity;

- searching for geological and hydrological publications, articles, and documents on the area under study from offices, departments, libraries and private firms;

- conducting orienting investigations. This involves plotting the locations of all existing shallow and deep wells on the topographical map. These wells and their locations should be checked in the field and an inventory made of other existing, but officially unknown, wells. If known, water quality data should be plotted on the topographical map and, depending on the data available, some hydrological cross-sections, showing the land surface and water table elevation should be drawn. This preparatory work may be of great help in drawing up an efficient and effective plan for the groundwater survey.

21.1.2 SURVEY OF GROUNDWATER LEVELS

With the preliminaries out of the way, the next step is to determine the extent and degree of the drainage problem, and here we begin by observing the water table (or phreatic surface).

As was mentioned in Chapter 1, Vol.I, agricultural land or land that is to be reclaimed is usually made up of a fine-textured layer whose thickness may vary from less than 1 m to several metres. Sandy material containing groundwater is often present underneath this layer and such groundwater may or may not be under pressure (semiconfined aquifer, Fig.1). If under pressure, the hydraulic head of the groundwater confined within this sand layer should be observed as well.

For a survey of groundwater levels, observations are made in:

- existing (village) wells
- open bore holes
- piezometers
- surface waters (lakes, streams, canals, etc.).

Existing wells

Existing wells offer ready-made sites for groundwater level observations. In most villages and on scattered farms or ranches one or more such wells can usually be found. Although their location may not always fit satisfactorily into the desired network of observation points (they may be sited on topographical heights only) it is most useful to include them in the network as they are cheap means of achieving our objective.

It should be noted, however, that a major problem of these wells is the validity of the observations obtained from them. If the well is hand-dug by the local farmer, we may be quite certain that the water level measured in the well indeed represents the water table. Because of its construction by hand the depth of these wells will seldom be more than about one metre below the lowest water table. Before the well is included in the network this should, of course, be checked. Hand-dug wells have a large diameter and thus a large storage capacity. This implies that it may take considerable time for the water level in the hole to adjust to changes of the water table, or to recover when water has been taken from it in substantial quantities, especially if the hydraulic conductivity of the water-bearing material is low. Water level readings taken shortly after water has been withdrawn from such a well are not valid.

A special problem arises if the existing well has been bored or drilled. This type of well usually has a much greater depth than a hand-dug well. It may penetrate several aquifers separated from one another by clay layers. Such wells are provided with a casing in which a perforated pipe or screen is fitted at the various depths where the aquifers occur. Other wells may have casings perforated from top

to bottom. The water level in these wells certainly does not represent the water table, but a composite of the various hydraulic heads occurring in the different aquifers. It must be emphasized therefore that if such wells are used the observations taken from them may not be valid. Before any existing well is included in the network, information on its depth, type of penetrated layers, casing and screen should be collected. Only then is it possible to conclude whether the well penetrates an unconfined aquifer (in which case it may provide valid information on the water table) or a semiconfined aquifer, or a multiple aquifer system (in which case it will provide no valid information on the water table).

Open bore holes

Open bore holes without any casing or screen can sometimes be used for watertable observations. These holes can easily be made with a hand auger as used in soil surveys, and may be 2 to 3 in. in diameter. The holes should be bored deep enough, i.e. a few decimetres below the lowest expected watertable depth. Care should be taken that the holes are provided with a reference point that remains in place and cannot easily be destroyed. Water table readings are taken from this point, hence its absolute elevation and elevation above the average land surface should be measured by a geodetic team. For this purpose a system of bench marks should be established in the area because contour lines of the land surface are values too rough to be used for accurate work.

Open bore holes are cheap means of measuring watertable depths. There are, however, a few troublesome factors. They can easily be destroyed by vandalism. The hole must therefore be hidden to the eye. Secondly, open bore holes may soon collapse, especially if they penetrate fine sands. Deepening the hole below the water table is often impossible because of the movement of these fine sands. In such circumstances, the hole can only be deepened by installing a casing and using a bailer to remove the sand. No heavy equipment is needed for this purpose; for holes 2 to 4 m deep, thick-walled plastic tubes 3 in. in diameter can be used as casing. Before the casing is removed, a plastic pipe, 1 or 2 in. in diameter and perforated over the lower 0.3 to 0.5 m, is installed inside the casing (see below).

Piezometers

Piezometers are open-ended pipes driven or jetted into the ground to the depth at which the hydraulic head is to be measured. The water level in the pipe corresponds with the hydraulic head at the pipe's lower end. The diameter of the pipe may vary from 1 to 3 in.

When piezometers are used, the following points should be taken into consideration. In a homogeneous unconfined aquifer, vertical flow components are usually lacking or of such minor importance that they can be neglected. Hence at any depth in an unconfined aquifer, the hydraulic head corresponds to the watertable height (phreatic level); in other words, in measuring the water table it does not make any difference how far the piezometer penetrates into the aquifer.

The hydraulic head of the water confined within a homogeneous semiconfined aquifer usually differs from the head of the phreatic water in the overlying confining layer. Since it is generally assumed that the flow of groundwater through a semiconfined aquifer is essentially horizontal and that vertical flow components can be neglected, the distribution of the head in such an aquifer is essentially the same everywhere in a vertical plane. The depth to which a piezometer penetrates into a semiconfined aquifer is therefore of little importance, although its bottom end should not be placed too close to either the overlying or underlying clay bed because of possible leakage effects.

If the hydraulic head in a semiconfined aquifer is higher or lower than the head of the phreatic water in the overlying semipervious layer, there is an upward or downward flow through this layer away from or towards the aquifer. The water level in a piezometer which penetrates into the semipervious covering layer is a function of its depth of penetration because the flow direction in this layer is mainly vertical (Fig.2). To measure the water table, the piezometer should therefore penetrate not more than a few centimeters into the saturated zone; but since water tables are usually changing during the course of the year, this procedure is not practical: if the water table is dropping the piezometer will soon fall dry. For practical purposes, piezometers are therefore, placed somewhat deeper, i.e. just below the lowest expected watertable depth.

To ensure that the piezometer functions properly, the lower 0.5 m of the pipe may be perforated. To prevent fine soil particles from entering the pipe or clogging the tiny holes, some jute or cotton is wrapped around the perforated part and the lower open end sealed with a plug. The pipe is then placed in the centre of a bore hole and, over the whole length of the perforated part, the annular space (space between pipe and wall of the hole) is filled with artificially graded fine gravel or coarse sand. This gravel pack facilitates the flow of water from the soil into the pipe and vice versa. The remainder of the annular space is then carefully filled with fine material (clay, clayey fine sand) whose permeability is low to prevent leakage along the pipe.

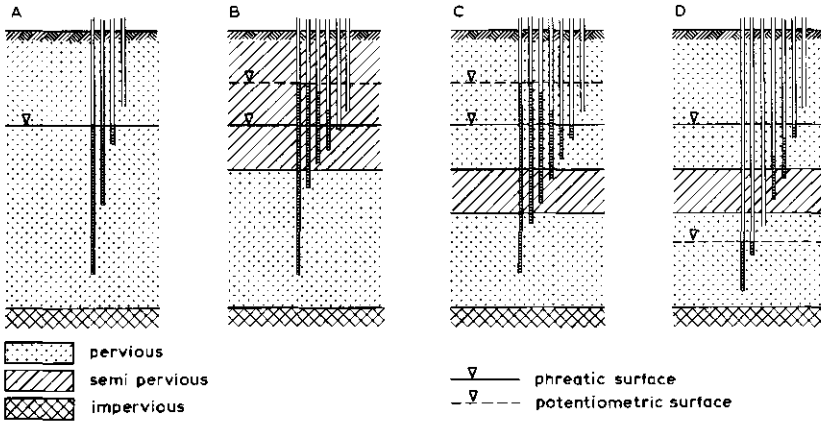


Fig.2. Hydraulic heads in piezometers installed at different depths in various types of aquifers and, where applicable, in their confining layers. A) an unconfined aquifer, B) a semiconfined aquifer, C) an unconfined aquifer overlying a semiconfined aquifer, and D) an unconfined aquifer overlying an other unconfined aquifer.

To keep the pipe in place and to prevent any direct contact between surface water and the backfill material, a concrete seal should be formed around the pipe, extending from the ground surface to a depth of a few decimeters. The top of the pipe is fitted with a screw cap provided with a tiny hole to allow air to escape (Fig.3).

Strictly speaking, a perforated pipe as described above cannot be called a piezometer; an observation well would be a better term. Since the perforations cover only the lower few decimeters of the pipe however, we will retain the term piezometer.

If piezometers are to be installed deeper than, say, 5 m, hand augers can no longer be used to bore the hole. Heavier boring equipment will then be required, although this need not be of the expensive sophisticated type. A simple tripod, cable, bailer, and 3 in. casings can make holes 10 to 20 m deep. If, for specific problems, still deeper holes are to be drilled, say 50 m or more, the same cable tool method can be applied, though heavier equipment will then be required. Unless sufficient information on the subsurface geological conditions is available, it is advisable to make a number of such deep (20 to 50 m) borings and provide them with one or more piezometers.

If an aquifer with intercalated fine textured layers is found at some depth, piezometers should be installed above and below these layers to determine any head

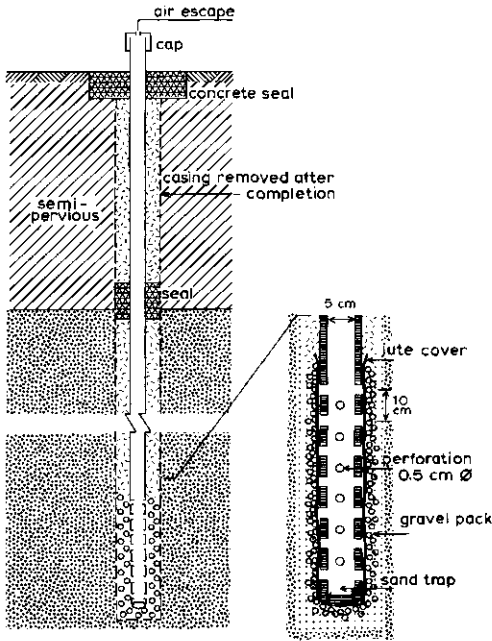


Fig.3. Schematic cross-section through a piezometer.

differences that may exist (Fig.4). Since the diameter of such deep borings is larger than that of hand borings, perforated clay layers should be properly sealed with bentonite or fine sandy loam to prevent leakage along the outside of the piezometer. If the diameter of the bore hole is sufficiently large, several piezometers can be placed at different depths in the same hole; here again it is essential that the piezometers be properly isolated.

Surface waters

A survey of the water table is incomplete if the levels of the surface waters that are in free connection with the groundwater are neglected. The water of a stream is either fed by the groundwater, the stream thus acting as a drainage channel (effluent stream), or the stream is feeding the groundwater, in which case it acts as a recharge channel (influent stream), see Figure 5.

Water levels of streams or other water courses represent local mounds or depressions in the water table and consequently are of great importance in a survey of groundwater conditions. A number of staff gauges should therefore be installed in these water courses and their absolute elevation measured.

If the water table lies below the bottom of the stream, the water level of the stream does not represent any point of the groundwater table. The stream is then losing water that percolates through the unsaturated zone to the deep water table.

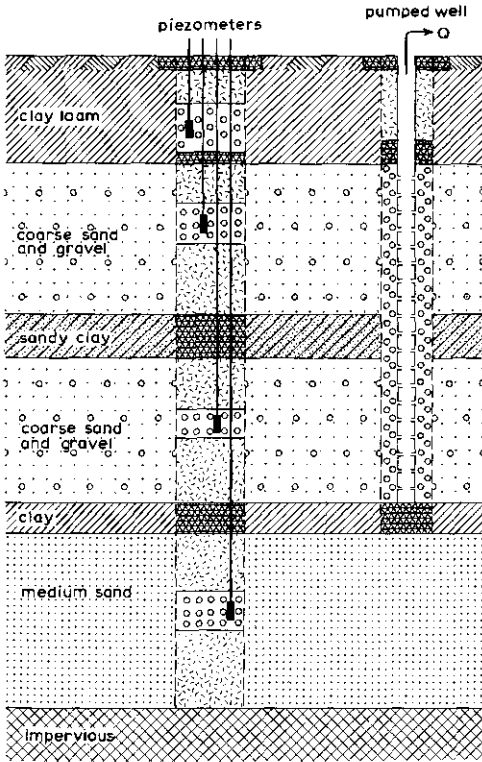
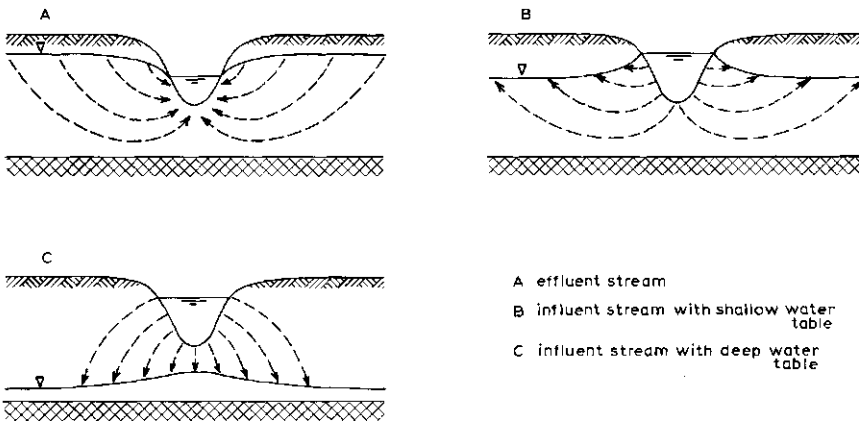


Fig.4. Multiple piezometer well



- A effluent stream
- B influent stream with shallow water table
- C influent stream with deep water table

Fig.5. Effluent (gaining) and influent (losing) streams

It thus recharges the groundwater system and a local mound is built up under the channel. In such a situation there is no need for a staff gauge in the channel but a piezometer should be installed on its banks.

21.1.3 OBSERVATION NETWORK

A network of observation points provides data on the elevation and variation of the water table and potentiometric surface.⁺⁾ These data can be used to determine:

- the configuration of the water table and the potentiometric surface
- the direction of the groundwater movement
- the location of recharge and discharge areas.

It will therefore be clear that in planning an observation network an optimal lay-out of the observation points i.e. one that provides maximum information at minimum costs, must be the aim. Too many points are as undesirable as too few, and what counts even more is their proper siting.

The lay-out of an observation network should therefore be based on any information on topography, geology, hydrology, soils, etc. collected and studied during the preparatory phase. Observation points will be required (Fig.6):

- along and perpendicular to lines of expected groundwater flow,
- at locations where changes in the slope of the water table (or the potentiometric surface) can be expected,
- in areas where significant changes in watertable elevation are likely to occur,
- on the banks of streams and other open water courses and along lines perpendicular to them, to determine the proper curving of the water table near such water courses,
- in areas where shallow water tables occur or can be expected in the future (for example in areas with a relatively high intensity of irrigation, or in seepage zones).

With regard to the density of the observation network, no strict rule can be given as this depends entirely on the topographical, geological, and hydrological conditions of the area under study, and on the type of survey (reconnaissance,detailed). As the required accuracy is generally inversely proportional to the size of

⁺⁾ The word *potentiometric* replaces the formerly used but incorrect term *piezometric* in such phrases as *piezometric head*, *piezometric surface*, in accordance with the recommendations made in U.S.Geol.Survey, Water Supply Paper No.1988, Lohman et al.

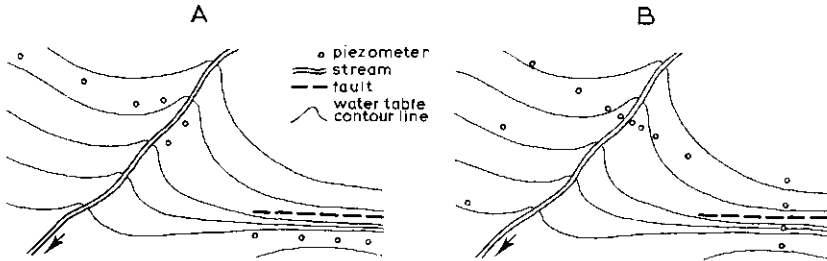


Fig.6. Lay-out of a piezometer network for expected groundwater conditions. A: incorrect, B: correct.

the area, the following relation may serve as a rough guide:

Size of area under study	number of observation points	number of observation points per 100 ha
100 ha	20	20
1,000 ha	40	4
10,000 ha	100	1
100,000 ha	300	0.3

The spacing of observation points may be increased as they are farther removed from lines of recharge or discharge (e.g. streams or canals) in approximately the following sequence: 10, 40, 100, 250, and 500 m, etc., with a maximum of 2000 to 3000 m.

As was mentioned in Chapter 6, Vol.I, groundwater flow problems cannot be solved unless it is known what happens at the boundaries of the flow system. An observation network must therefore extend somewhat beyond the boundaries of the area under study so as to determine qualitatively and quantitatively any inflow from and outflow to adjacent areas.

21.1.4 WATER LEVEL READINGS

Water level measurements are always taken from a reference point, for example, the rim of a piezometer or, if hand-dug wells are used, a mark on the wall of the hole. The height of the reference point above or below the ground surface and its absolute elevation above or below mean sea level must be known. All water level observation data can then be converted into:

- depth below ground surface
- absolute elevation above or below mean sea level.

Equipment

Water level measurements can be taken in various ways:

- *Wetted tape method*: A chain or steel tape (with calibration in millimetres) to which a weight is attached, is lowered into the pipe or bore hole to below the water level. The lowered length of chain from the reference point is noted. The chain is then pulled up and the length of its wetted part measured (this is facilitated by chalking the lower part of the chain). When the wetted length is subtracted from the total lowered length, this gives the depth to the water level below the reference point (Fig.7A).

- With a *mechanical sounder*, consisting of a small piece of steel or copper tube (0.5 to 1 in. in diameter and 2 to 3 in. long) which is closed at its upper end and connected to a calibrated steel tape or to a chain. When lowered into the pipe it produces a characteristic sound upon hitting the water. The depth to the water level can be read directly from the steel tape or measured afterwards along the chain (Fig.7B).

- With an *electric water level indicator*, consisting of a double electric wire with two electrodes at the lower ends. The upper ends of the wire are connected to a battery and an indicator device (lamp, mA meter, sounder). When the wire is lowered into the pipe and the electrodes touch the water, the electrical circuit closes, which is shown by the indicator. If the wire is calibrated, the depth to the water level can be read directly (Fig.7C).

- With a *floating level indicator or recorder*, consisting of a float and counterweight attached to an indicator or recorder (Fig.7D). Recorders can generally be set for different lengths of observation period. They require, however, relatively wide pipes. New developments are recorders that punch a water level code in a paper tape (the paper tape can be fed directly to an electronic computer for processing) and recorders equipped with a radio emitter that at given times automatically signals the water level to a receiving post at a field office.

- The water levels of open water surfaces are usually read from a staff gauge or a water level indicator installed at the edge of the water surface.

Frequency of measurements

The frequency at which water level measurements should be taken depends on the type of study. In a reconnaissance survey, a frequency of once or twice a month will generally be sufficient. To obtain a representative picture of the position of the water table (and potentiometric surface) in the area under study, all the

measurements should as far as possible be taken on the same date, for example, on the 14th and 28th of each month. If this proves impossible for whatever reason, the water level of the particular date may, under certain conditions, be estimated by (graphical) interpolation.

If special problems are to be investigated, for example, the effect of heavy rainfall or excess irrigation on the water table, the frequency of measurements should be increased to, say, once every hour. If possible an automatic recorder should also be installed on a representative well. Other problems that require frequent measurements are: instantaneous or gradual rise or fall of the water level in open water courses and their effect on the water table in adjacent land, the transmission of tidal movement in adjacent land (Chap.13, Vol.II), and the effect on the water table when a well is pumped (Chap.12, Vol.II).

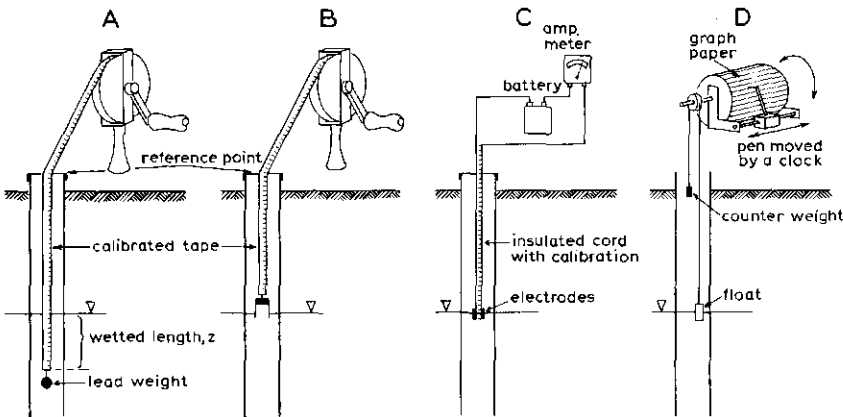


Fig.7. Methods used to determine depth to water in wells and piezometers. A: wetted tape, B: mechanical sounder, C: electric water level indicator, and D: automatic recorder.

21.1.5 BASE OBSERVATION NETWORKS

Once an observation network has been established, water level measurements should continue for at least a few years, preferably including both wet and dry years. When sufficient readings are available, the hydrographs of all the observation points should be systematically analysed. A comparison of these hydrographs enables us to distinguish different groups of wells and piezometers. Each well or piezometer belonging to a certain group shows a similar response to the recharge and discharge pattern of the area. By a similar response we mean that the water level in these wells and piezometers starts rising at the same time, attains its maximum value at the same time, and, after recession starts, reaches its minimum value at the same time. The amplitude of the water level fluctuation in the

various wells need not necessarily be the same but should show a great similarity. Areas where such wells and piezometers are sited can then be regarded as hydrological units.

The water level readings of a certain well in a group of wells can be correlated with those of another well in that group. If the two wells correlate satisfactorily, one of the two can be dropped from the network. Such an analysis may lead to the selection of a number of standard wells only, and the network can thus be reduced. From the water level readings in these standard wells, which form the base network, the water level in the other wells that were dropped can be calculated from the established correlation. Another way of reducing the amount of measuring work is to reduce the frequency of observation in a number of wells; for example, measurements can be taken once every season instead of twice a month.

To calculate the correlation of two wells or piezometers, the method of linear regression is used. An advantage of this technique is that gaps in the readings can be filled by calculation. Such gaps may occur because the well was destroyed and could not be repaired promptly, or because it fell dry for some months.

The procedure of the linear regression method is as follows:

- Select two wells or piezometers showing a similar trend in water levels (Fig.8A) and draw a fluctuation diagram by plotting the water levels of one well against those of the other (Fig.8B). Prepare the first four columns of Table 1.

- Calculate $\bar{x} = \frac{\sum x}{n}$ and $\bar{y} = \frac{\sum y}{n}$ and prepare the other columns of Table 1.

- Calculate $s_x = \sqrt{\frac{\sum(\Delta x)^2}{n-1}}$ and $s_y = \sqrt{\frac{\sum(\Delta y)^2}{n-1}}$

- Calculate the correlation coefficient $r = \frac{\sum(\Delta x \cdot \Delta y)}{(n-1)s_x \cdot s_y}$

The correlation coefficient should be at least 0.85

- Calculate the regression coefficient $\alpha = r \frac{s_y}{s_x}$

- Substitute the values of \bar{x} , \bar{y} and α into the equation of the regression line

$$(y - \bar{y}) = \alpha (x - \bar{x})$$

which gives the equation

$$y = \alpha x + b \tag{1}$$

- Select at random two values for y and calculate with Eq.1 the corresponding values of x . Plot the points (x_1, y_1) and (x_2, y_2) on the fluctuation diagram and draw a straight line through these points. This line is the graphical representation of Eq.1.
- The water levels of one of the wells can now be calculated by substituting water level data of the other well into the regression equation. So one of the two wells can be dropped from the network.

A numerical example is given below. Figure 8A shows the well hydrographs of the observation wells x and y . Figure 8B is the fluctuation diagram. In Table 1 the water level readings are transformed into the parameters required for the quantification of the regression equation.

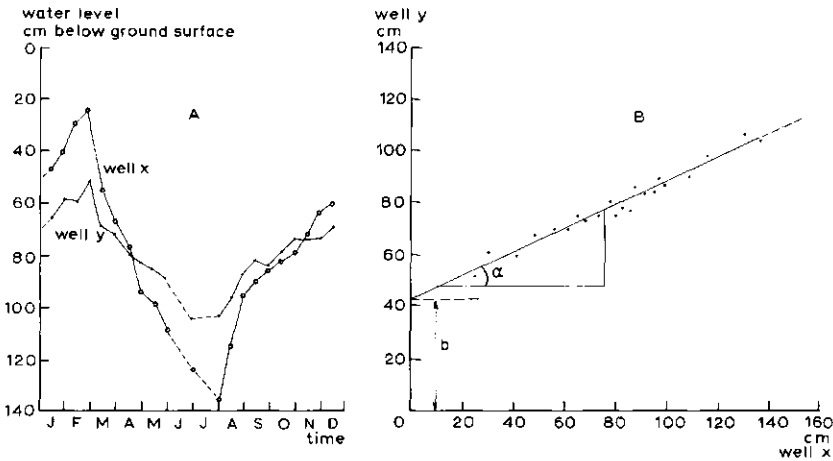


Fig.8. A: Hydrographs of observation wells x and y . B: Fluctuation diagram of these wells showing the relationship between their water levels relative to a datum plane.

Substitution of these parameters into the appropriate equations yields:

$$s_x = \sqrt{\frac{\sum(\Delta x)^2}{n-1}} = \sqrt{\frac{18249.16}{20}} = 30.207 \quad s_y = \sqrt{\frac{\sum(\Delta y)^2}{n-1}} = \sqrt{\frac{3893.84}{20}} = 13.953$$

$$r = \frac{\sum \Delta x \cdot \Delta y}{(n-1)s_x \cdot s_y} = \frac{8300.88}{20 \times 30.207 \times 13.953} = 0.985$$

$$\alpha = r \frac{s_y}{s_x} = 0.985 \times \frac{13.953}{30.207} = 0.455$$

and

$$y - 78.2 = 0.455 (x - 79.4)$$

or

$$y = 0.455 x + 42.1$$

Note that:

- The number of simultaneous observations (n) should be at least 20; they should cover both wet and dry periods.
- The related wells should be located in areas with approximately similar hydrological conditions and they should possess about the same fluctuation range.
- Wells located in areas with spot-wise irrigation, or in areas subject to heavy pumping or to unequal rainfall patterns, cannot be related.
- Wells to be dropped from the observation network should have been correlated with several other wells of the hydrologic unit area to which they belong.

21.1.6 WATER QUALITY MEASUREMENTS

A survey of groundwater conditions forming part of a drainage survey also involves the groundwater chemistry. Although the details of groundwater chemistry may be exceedingly complex, natural variations in groundwater quality should not be considered random; they are geohydrologically controlled. Hence, to understand these variations one must have a certain knowledge of the geologic history and hydrodynamics of the groundwater system.

A survey of groundwater quality is not merely a matter of determining the existing state. It also involves a study of these natural variations in groundwater quality, why they exist, and how they may change if drainage works are carried out. Lowering a water table by artificial drainage means that groundwater from adjacent lands and from deep layers may start moving to the drained land. If that groundwater is salty, the drained area and its surface water system will then be charged daily with considerable amounts of dissolved salts. Hence it is important to gain an idea of the quality of this groundwater, deep as well as shallow.

Natural variations in groundwater quality are related to the geomorphology of the terrain. Even minor relief features, as are often found on alluvial plains, usual-

ly have a clear effect on groundwater quality. Relatively high-lying topographic-
al sand ridges, for example, are areas of recharge and consequently contain
groundwater of excellent quality. On its way to adjacent topographical depres-
sions the water may gradually be mineralized. Hence in these low-lying areas,

Table 1. Transformation of readings of wells x and y

Rank number	date	x	y	$x-\bar{x}$ = Δx	$y-\bar{y}$ = Δy	$(\Delta x)^2$	$(\Delta y)^2$	$\Delta x \cdot \Delta y$
1	14/1	48	66	-31.4	-12.2	985.96	148.84	383.08
2	30/1	41	59	-38.4	-19.2	1474.84	368.84	737.28
3	14/2	30	60	-49.4	-18.2	2440.06	331.24	899.08
4	28/2	25	51	-54.4	-27.2	2959.36	739.84	1479.68
5	14/3	56	69	-23.4	- 9.2	547.56	84.64	215.28
6	30/3	68	72	-11.4	- 6.2	129.96	38.44	70.68
7	14/4	78	79	- 1.4	0.8	1.96	0.64	- 1.12
8	30/4	95	83	15.6	4.8	243.36	23.04	74.88
9	14/5	99	86	19.6	7.8	384.16	60.84	152.88
10	30/5	109	89	29.6	10.8	876.16	116.64	319.68
	14/6							
11	30/6	130	105	50.6	26.8	2560.36	718.24	1356.08
	14/7							
12	30/7	136	103	56.6	24.8	3203.56	615.04	1403.68
13	14/8	116	97	36.6	18.8	1339.56	353.44	688.08
14	30/8	97	88	17.6	9.8	309.76	96.04	172.48
15	14/9	91	82	11.6	3.8	134.56	14.44	44.08
16	30/9	87	85	7.6	6.8	57.76	46.24	51.68
17	14/10	83	78	3.6	- 0.2	12.96	0.04	- 0.72
18	30/10	80	74	0.6	- 4.2	0.36	17.64	- 2.52
19	14/11	73	74	6.4	- 4.2	40.96	17.64	26.88
20	30/11	65	74	-14.4	- 4.2	207.36	17.64	60.48
21	14/12	61	69	-18.4	- 9.2	338.56	84.64	169.28
	30/12							
		1668	1643	0.6	- 0.8	18249.16	3893.84	8300.88

$$\bar{x} = \frac{\sum x}{n} = \frac{1668}{21} = 79.4; \quad \bar{y} = \frac{\sum y}{n} = \frac{1643}{21} = 78.2$$

groundwater of much poorer quality is usually found. There the water table is usually very shallow and, especially in arid climates, a considerable capillary transport of groundwater to the land surface occurs, increasing the process of salinization with high evaporation rates.

Salinization of groundwater may also be man-made, for example in irrigated areas which have inadequate drainage. The salts washed out from the soil during irrigation may reach the groundwater, which thus becomes more and more mineralized if natural drainage is too slow or nonexistent. In such a situation a zonation in groundwater quality may come into being, i.e. the shallow groundwater to a depth of some metres may be very saline, whereas the deeper water is less saline or only slightly brackish or in some cases even fresh. Zonations in groundwater quality may, however, also be due to natural causes (changes in depositional environment due to rise and fall of the sea level; Chap.1, Vol.1).

An adequate knowledge of groundwater quality is, of course, also required if wells are to be used to drain land and the pumped water is to be used for irrigation of crops.

Electrical conductivity

The total concentration of soluble salts in groundwater can, for purposes of diagnosis and classification, be adequately expressed in terms of electrical conductivity (EC). The conductivity can readily and precisely be determined in the field with a portable conductivity meter. Once the network of observation wells, bore holes, and piezometers has been established, water samples should be withdrawn from all of them to measure the electrical conductivity.

To ensure that the water sample is representative of the groundwater near the well or piezometer, water should be withdrawn by means of a small bailer or pump. When a sufficient quantity of water has been removed from the well and the water is clean, a sample is taken.

Major chemical constituents

The electrical conductivity expresses the total concentration of soluble salts in the groundwater, but it gives no information on the types of salts that are present. The major constituents Ca^{++} , Mg^{++} , Na^+ , K^+ , CO_3^{--} , HCO_3^- , SO_4^{--} , and Cl^- can only be determined in the laboratory.

Since these chemical analyses are costly, not all the wells, bore holes, and piezometers should be sampled for detailed analysis. From the results of the

electrical conductivity measurements, certain wells should be selected for detailed analysis. These analyses allow other chemical characteristics of the water to be calculated (SAR and "residual sodium carbonate").

Sodium adsorption ratio

Soil in contact with water that contains sodium may absorb this constituent from the water. If exchangeable sodium is accumulated, alkali soils characterized by poor tilth and low permeability, will form.

The sodium-adsorption-ratio is generally used as an index of the sodium or alkali hazard of the water and is defined by the equation:

$$\text{SAR} = \text{Na}^+ / \sqrt{(\text{Ca}^{++} + \text{Mg}^{++})/2} \quad (2)$$

where Na^+ , Ca^{++} , and Mg^{++} represent the concentrations in milliequivalents per litre of the respective ions.

Classification of waters

Groundwater can be classified in different ways. From a hydrogeological point of view, groundwater is usually classified on the basis of the dominant ions present. An example is given in Figure 9. In this diagram the chemical composition of a water is given as percentage of equivalents per million of the anions and cations and is plotted as a single point. Waters that plot in Field I are calcium bicarbonate waters (secondary alkalinity); in Field II sodium carbonate or bicarbonate water (primary alkalinity); in Field III sulphate, chloride, calcium, magnesium water (secondary salinity); in Field IV sulphate, chloride, sodium water (primary salinity); and in Field V no one of the cation-anion pairs exceeds 50 per cent. A classification of groundwater which is more relevant for drainage and irrigation purposes is based on the above-mentioned values of SAR and EC. The diagram used for classification is that given by RICHARDS (1954; see Fig.10). Salinity (C) and sodium (S) are classed in four groups - low, medium, high, and very high - so that 16 water types can be distinguished, varying from excellent (C₁S₁) to very poor (C₄S₄).

It should be noted that these and other classifications should be used with caution and should never be applied too rigidly. The salt tolerance level of different crops varies considerably, and so does the drainability of the soil. Irrigation techniques also differ. If drip irrigation is applied, a much higher salinity can be accepted than the commonly used maximum EC value of 2250 or 2500 micromhos/cm. Hence groundwater quality should be considered in relation to crops, soil drainability, and the irrigation technique applied.

Residual sodium carbonate

When much bicarbonate is present in the water, Ca^{++} and Mg^{++} tend to precipitate as carbonates if evapotranspiration causes the soil solution to become more concentrated. The relative concentration of sodium increases and, as a result, adsorption of sodium to the soil complex is likely to increase. The equation expressing the residual sodium carbonate reads

$$\text{residual Na}_2\text{CO}_3 = (\text{CO}_3^{=} + \text{HCO}_3^-) - (\text{Ca}^{++} + \text{Mg}^{++}) \quad (3)$$

where the ionic constituents are expressed as milliequivalents per litre. Waters containing more than 2.5 meq/l residual sodium carbonate are not suitable for irrigation purposes. Waters with 1.25 to 2.5 meq/l are marginal, and those containing less than 1.25 meq/l are probably safe (RICHARDS,1954).

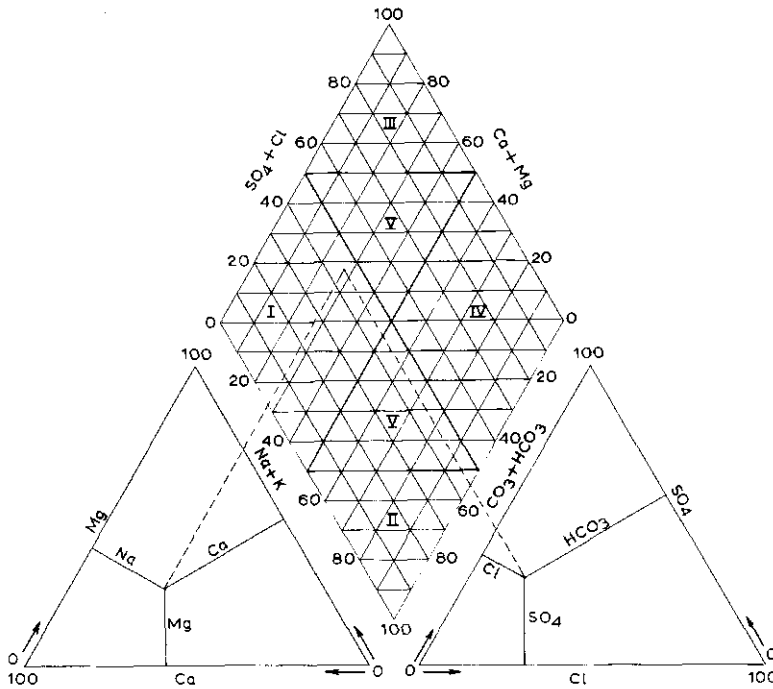


Fig.9. Geochemical classification of groundwaters. After PIPER (1944).

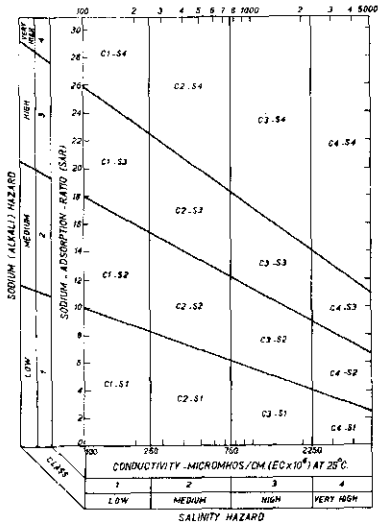


Fig.10. Diagram for the classification of irrigation waters. After RICHARDS (1954).

21.2 PROCESSING OF GROUNDWATER DATA

Processing of groundwater data comprises:

- compilation of the data on special forms (groundwater records)
- plotting of the water levels against time (well hydrographs)
- presenting water level and water quality data in map form (groundwater maps).

21.2.1 GROUNDWATER RECORDS

The readings for each observation point are entered on a waterlevel record form, an example of which is shown in Figure 11. Recorded for each observation are: date of observation, observed depth of the water level below the reference point (reading), calculated depth below ground surface (for phreatic levels only), and calculated water level elevation. Also mentioned on the form are the location and the relevant characteristics of the observation point and the soil profile.

The record should be kept up-to-date. In this way any obvious error in the readings can immediately be detected and, if need be, a new reading made. Groundwater level records contain irreproducible observations, so the forms should be carefully filed.

Fig.11. Example of a water level record form.

GENERAL DATA OBSERVATION POINT NR.....

MAP NR..... COORDINATES: X = Y =

MUNICIPALITY..... PROVINCE

OWNER..... INSTALLATION DATE..... TYPE¹.....

DEPTH..... SCREENED PART..... AQUIFER TYPE².....

WELL LOG: FILE NR..... WATER SAMPLES: FILE NR.....

SURFACE ELEVATION..... REFERENCE POINT ELEVATION.....

OBSERVATIONS

DATE	READING ³	ELEVATION ⁴	DEPTH ⁵	REMARKS ⁶

¹ e.g. village well, open bore hole, piezometer
² e.g. unconfined aquifer, semiconfined aquifer, semipervious covering layer
³ with respect to reference point
⁴ with respect to general datum, for example mean sea level
⁵ below ground surface (for phreatic levels only)
⁶ data on water samples, irrigation, water at the surface, flow from wells, water withdrawal (pumping), etc.

21.2.2 WELL HYDROGRAPHS

To obtain a well hydrograph (Fig.12), the water level readings for each observation point are plotted against time.

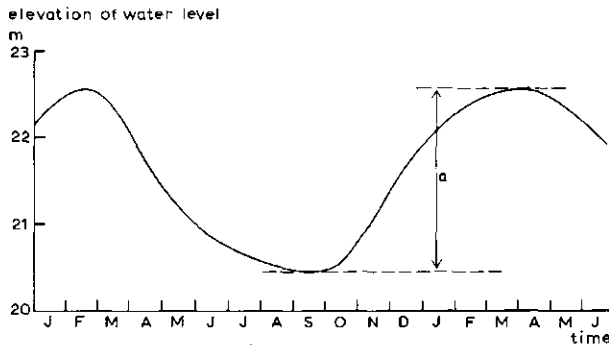


Fig.12. Well hydrograph showing annual sinusoidal fluctuation of water level (a = amplitude).

Well hydrographs are fundamental in evaluating groundwater conditions as they provide many items of information.

They show

- the rate of rise or fall of the water level;
- hydrographs showing the depth of the water table below ground surface can reveal the periods of the year when critical water tables occur;
- in combination with relevant information on water balance components (precipitation, irrigation, pumping from wells for irrigation, etc.) a hydrograph may help one to understand the cause of observed water table fluctuations;
- hydrographs covering several years give an indication of the long term trends in groundwater behaviour: general rise or fall of the water table, i.e. replenishment or depletion of the aquifer;
- hydrographs, when arranged according to their similarity, enable areas with a uniform groundwater behaviour to be delimited (hydrologic units);
- short term fluctuations, related to pumping from a well or the changing water level in open water courses, may be used to calculate the hydraulic properties of the aquifer (Chaps.12 and 13, Vol.II).

21.2.3 GROUNDWATER MAPS

An area's groundwater conditions can best be analyzed by plotting the groundwater data on maps. The following maps are the most useful:

- contour maps of the water table and the potentiometric surface,
- depth-to-water-table maps,
- groundwater-fluctuation maps (water table and potentiometric surface),
- maps showing the difference in head between the water table and the potentiometric surface,
- groundwater quality maps.

Watertable contour maps

A watertable contour map is a map of the phreatic surface; it can be prepared for a specific date, but preferably as a mean for a longer period (a season or a year).

The maps are prepared by plotting the absolute water levels of all observation points on a topographical map and drawing lines connecting points of equal watertable elevation (equipotential lines, or isohypses, or watertable contour lines). Such maps enable the configuration of the phreatic surface and the direction and intensity of the groundwater flow to be studied (see Section 3.2).

To draw contour lines, water levels between the observation points must be interpolated. This can be done either by eye or by linear interpolation (Fig.13). The linear interpolation method may cause serious errors if the influence of the topography and of geological structures is ignored (Fig.14). Therefore contour lines should preferably be drawn free hand, using all available topographic and geological information.

Depth-to-water-table maps

A depth-to-water-table map is best derived from the difference in elevation between the contour lines of the ground surface and those of the water table. A map showing the depth of the water table below the ground surface is of special use in delimitating the extent of areas in need of drainage. For that reason depth-to-water-table maps are drawn for some chosen critical dates, for example, when the highest water table occurs (deduced from the well hydrographs), when certain farming operations have to be carried out, or when the crops are expected to be most sensitive to high water tables.

Groundwater fluctuation maps

Groundwater fluctuation maps are constructed by plotting for a given span of time the change of the water level in the observation wells and drawing lines of equal change. These maps are constructed either by plotting the difference in water

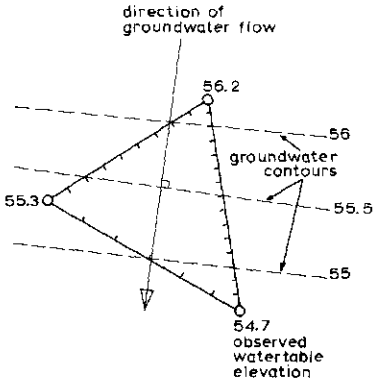
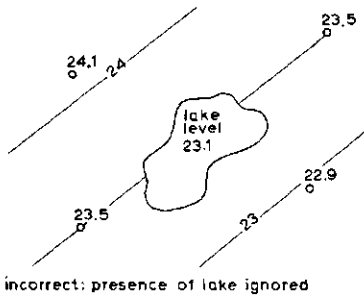
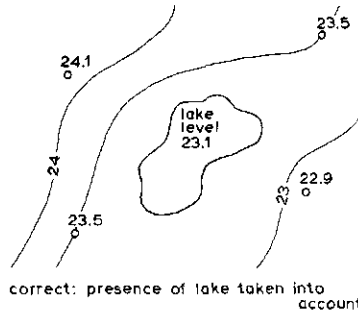


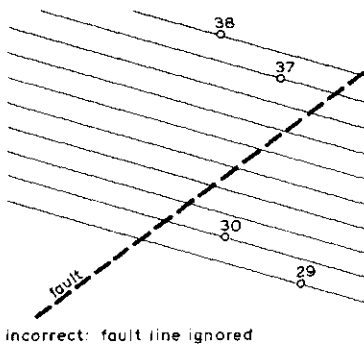
Fig.13. Construction of watertable contour lines by linear interpolation.



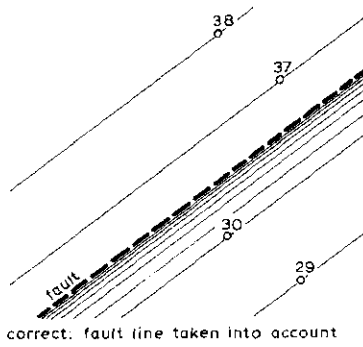
incorrect: presence of lake ignored



correct: presence of lake taken into account



incorrect: fault line ignored



correct: fault line taken into account

Fig.14. Two examples of correctly and incorrectly drawn watertable contour lines.

level in the observation wells or by superimposing two watertable contour maps and plotting the difference in water level at contour intersections (Fig.15). This type of map can be drawn both for fluctuations in the elevation of the water table and for fluctuations in the elevation of the potentiometric surface of a confined or semiconfined aquifer.

Hydraulic-head-difference maps

In areas underlain by a semiconfined aquifer, double piezometers (one shallow piezometer in the upper unconfined covering layer and one deep piezometer in the aquifer) may show different water levels. If the potentiometric head in the aquifer is higher than the groundwater level in the upper unconfined layer, the latter is recharged from the aquifer; if the potentiometric head is lower than the groundwater level in the upper layer, this layer loses water to the underlying aquifer.

In areas underlain by a semiconfined aquifer it is useful to construct a hydraulic-head-difference map. This can be done either by plotting for a given date the difference in water level observed in double piezometer wells and drawing lines of equal head difference, or by superimposing for a given date the contour maps of the water table and the potentiometric surface and plotting the head difference at contour intersections. Lines of equal head difference can then be drawn.

Groundwater quality maps

One of the basic maps to be drawn is an electrical conductivity map of the shallow groundwater. If data from deep observation wells and piezometers are available, an electrical conductivity map of the deep groundwater should be drawn too. By plotting all the EC values on a map, lines of equal electrical conductivity (equal salinity) can be drawn. Preferably the following limits should be taken (see also Fig.10): less than 100 micromhos/cm; 100 to 250; 250 to 750; 750 to 2500; 2500 to 5000; and more than 5000. Other limits may, of course, be chosen, depending on the salinity found in the waters.

The next map to be drawn is that showing the distribution of the sodium adsorption ratio of the groundwaters. The SAR values are plotted on a map and lines of equal SAR are drawn. Preferably the following limits should be used (see also Fig.10): less than 10; 10 to 18; 18 to 26; and more than 26. Other limits may also be chosen if the SAR values do not allow the use of the above limits.

The EC and SAR values are used to classify the groundwater (Fig.10). The groundwater class is then plotted on a map and class boundaries are drawn, delimitating

areas where high, medium, and low electrical conductivity occurs and where, and to what extent, a sodium hazard exists.

If the groundwater in most of the area under study is saline, it is also useful to draw maps showing the distribution of the chloride and sulphate contents of the groundwater.

The accuracy of groundwater maps depends, amongst other things, on the number of observation points and their distribution. For this reason each map, whether a water table map or a water quality map, should clearly show the measuring points on which it is based. Wells not used for the specific purpose should be dropped from the map or should be marked differently.

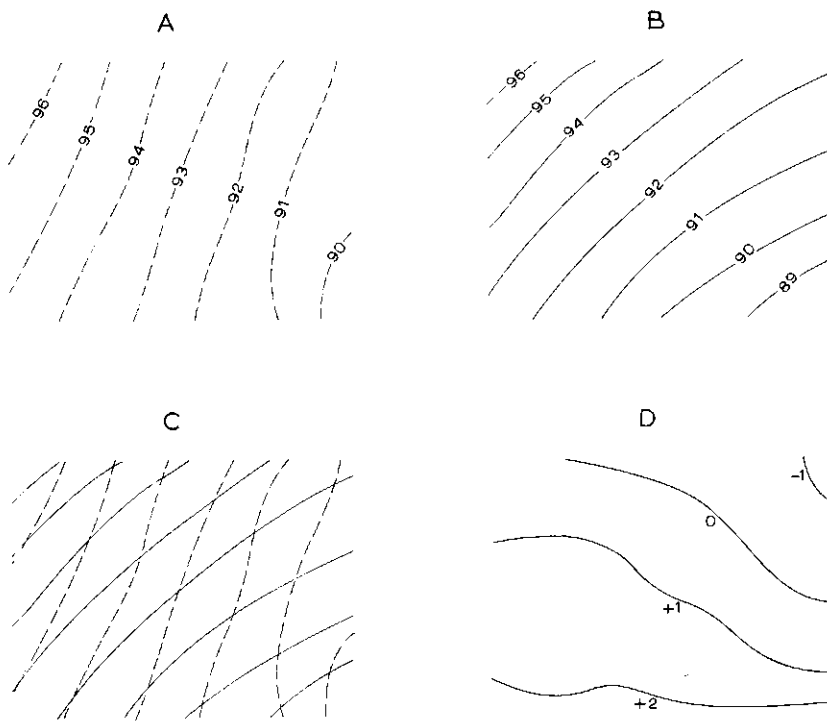


Fig.15. Construction of waterlevel fluctuation map by superimposing watertable contour maps of two hydrologically different years. A: present water levels, B: past water levels, C: Maps A and B superimposed, and D: resulting waterlevel fluctuation map.

21.3 EVALUATION OF GROUNDWATER DATA

21.3.1 EVALUATION OF HYDROGRAPHS

Water level changes can be classified into two kinds:

- changes due to changes in groundwater storage
- changes due to other influences, e.g. changes in atmospheric pressure, deformation of the aquifer, disturbances within the well.

Most fluctuations in water levels are caused by changes in storage. It is these changes that we are primarily interested in because they are the net result of the prevailing recharge and discharge pattern. Under undisturbed natural conditions there will usually be an annual equilibrium of recharge and discharge. Rising water levels indicate the periods when recharge is exceeding discharge and declining water levels the periods when discharge is exceeding recharge (Fig.16). Rather abrupt changes in the amount of water stored (Fig.17) will be found near stream channels influenced by the rise and fall of the surface water level and in relatively shallow phreatic aquifers under the influence of precipitation or irrigation. Although the effect of precipitation on the water table is usually quite clear, an exact correlation is often difficult because:

- differences in effective porosity will cause the water table to rise unevenly
- part of the precipitation may not reach the water table at all because it is discharged as surface runoff and/or stored in the unsaturated zone above the water table
- the groundwater flow terms may result in a net groundwater recharge or a net groundwater discharge, affecting the water table position.

Evapotranspiration on the other hand may cause a lowering of a shallow water table when the resulting water losses exceed the net groundwater recharge.

Pumping causes local changes in storage. The effects of pumping depend on the length of the pumping period and on the pumping rate. More or less continuous pumping for irrigation purposes during a large part of the year may have a residual influence many months after pumping has stopped. In areas where pumping exceeds replenishment the recovery will not be complete before the next pumping period starts and a gradual lowering of the groundwater table will result (Fig.18).

After studying each hydrograph and relating the water level changes to the various causes of recharge and discharge, the hydrographs can be arranged in a number of

groups in conformity with their fluctuation pattern. Each group represents an area where the changes in water table or potentiometric surface are related in the same way to the recharge and discharge components of the system. From the hydrographs of each group of piezometers and bore holes, we can read what are the critical periods of the year in which the water table is too high and needs to be controlled by artificial drainage. Further we can find from them what components of recharge (precipitation, water losses from canals, field irrigation losses, subsurface inflow) the groundwater system is most sensitive to. Hence the effects of changes in the recharge and discharge pattern may be predicted.

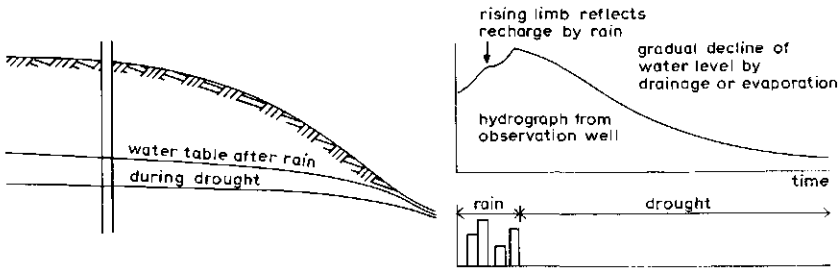


Fig.16. Hydrograph showing rise of water table during recharge by rain and the subsequent decline during drought.

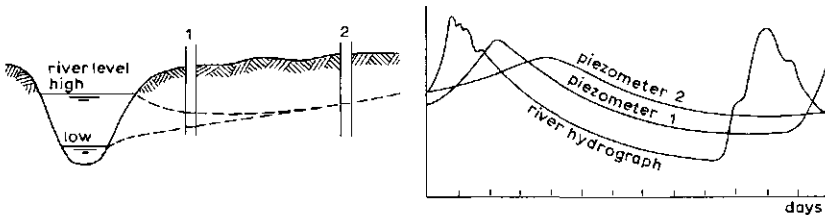


Fig.17. Influence of fluctuations of the water level in a river on the water table in adjacent land. Note that the influence diminishes with increasing distance to the river.

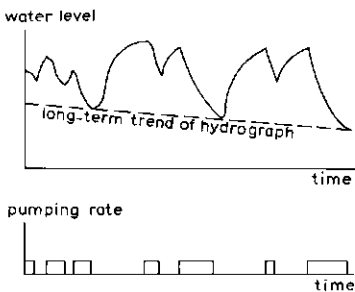


Fig.18. Hydrograph of a piezometer in a pumped aquifer, and corresponding pumping pattern.

21.3.2 EVALUATION OF GROUNDWATER CONTOUR MAPS

The fundamental importance of the water table and the potentiometric surface in groundwater investigations that form part of a drainage study may be outlined by the following brief summary.

Flow direction. The contour lines of a water table map or a potentiometric surface map are in fact equipotential lines (Chap.6, Vol.I). Hence the direction of the groundwater flow, being perpendicular to the equipotential lines, can be directly deduced from these maps (Fig.19).

Hydraulic gradient ($dh/dx = i$). A water table contour map or a contour map of the potentiometric surface of a semiconfined aquifer is a graphic representation of the hydraulic gradient of the water table or potentiometric surface. The hydraulic gradients, which can be directly derived from these maps, are the basis for calculating the rate of groundwater flow through cross-sections, for example, through project area boundaries. The flow velocity (v) varies directly with the hydraulic gradient and, at constant flow velocity, the gradient is inversely related to the hydraulic conductivity (K), or $v = Ki$ (Darcy's law). This is a fundamental law governing the interpretation of hydraulic gradients of water tables or potentiometric surfaces. If the flow velocity in two cross-sections of equal depth and width is the same, but one cross-section shows a greater hydraulic gradient than the other, its hydraulic conductivity must be lower. On the other hand, small hydraulic gradients (flat watertable slopes) reflect a high hydraulic conductivity of the water-transmitting material. Hence if in a certain flow direction the spacing of the contour lines is narrowing (hydraulic gradients becoming greater) the hydraulic conductivity of the material becomes lower.

Groundwater mounds. A mound in the water table or potentiometric surface is usually caused by recharge, either from above (for example, by local irrigation), or from below (for example, by a local upward groundwater flow from deep layers - see Fig.20). If a local mound in the water table is not due to recharge from irrigation or some other surface source of local nature, we can be sure that it is caused by the upward flow of groundwater from deep layers. Typical examples can be found in alluvial plains underlain by karstic limestone containing water under pressure. This water may escape locally through cracks or karstic holes, giving rise to mounds in the water table or potentiometric surface.

Groundwater depressions. A local depression in the water table is usually due to pumping from wells (Chap.12, Vol.II). If no such pumping occurs and if karstic limestone exists under the alluvium, groundwater may be lost locally through cracks or holes in the limestone, causing local depressions in the water table.

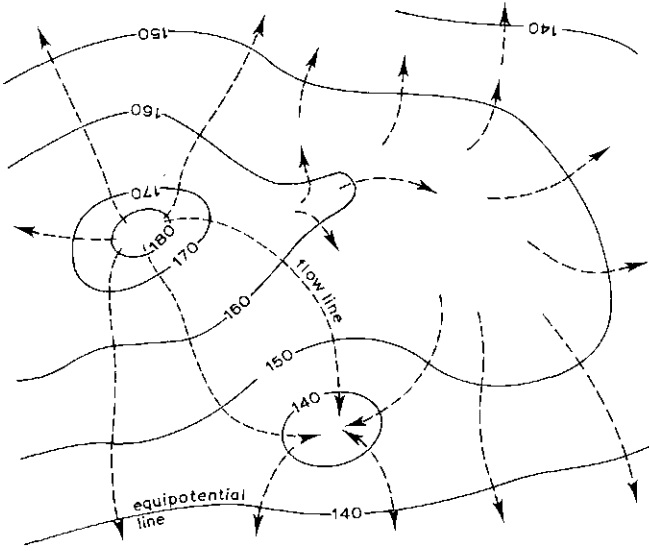


Fig.19. Pattern of equipotential lines and flow lines.

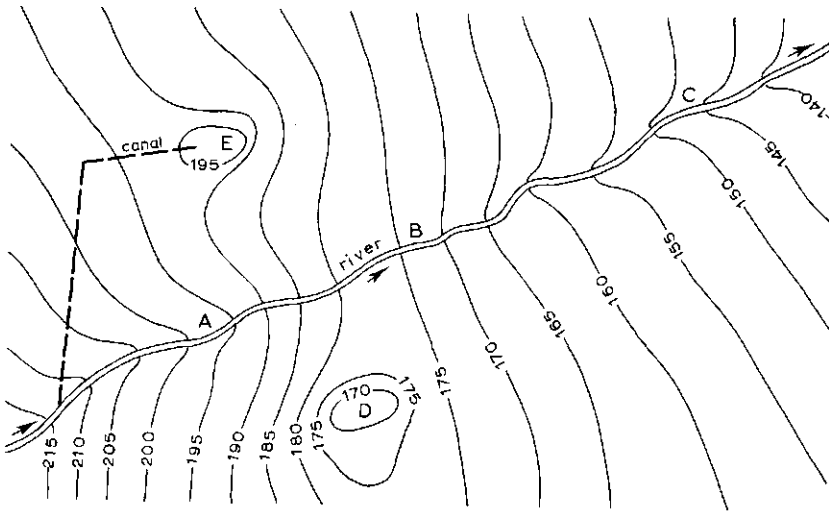


Fig.20. Water table contour map. A: area of recharge, river influent (losing); B: water table elevation is the same as water level in the river; C: area of discharge, river effluent (gaining); D: area of discharge, pumped wells; E: area of recharge, deep percolation of irrigation water.

Influent or effluent streams. The curvature of contour lines of the water table in the vicinity of streams or other surface water bodies indicates whether it is an influent (losing) or an effluent (gaining) stream. For streams losing water, the contour lines bend downstream and for streams gaining water they bend upstream (Fig.20).

Radial resistance. The bends in contour lines near effluent streams (drainage channels) may have different shapes due to differences in the radial resistance of the stream. The concept of radial resistance was introduced by ERNST (1962) as the resistance that the groundwater has to overcome while flowing into the stream bed due to the contraction of the flow lines in the vicinity of the stream. Long and narrow bends indicate a high radial resistance; short and wide bends a low radial resistance. The radial resistance can be determined from the hydraulic gradients at some distance on either side of the stream bed (i_1 and i_2) where the flow is mainly horizontal. If furthermore the transmissivity of the aquifer (KD) is known, the value of this resistance can be found from the following expression:

$$q_o = (i_1 + i_2) KD = \frac{\Delta h_w}{w} \quad (4)$$

where

- q_o = total outflow in a vertical section through the wetted perimeter of a stream or channel (m^2/day)
- w = radial resistance of the stream or channel (day/m)
- Δh_w = head difference, difference between water level in the stream and the water table elevation (Fig.21), (m)
- KD = transmissivity of the water bearing layer (m^2/day)
- i_1 and i_2 = water table gradient on the left and right bank, respectively, at some distance from the stream where the groundwater flow is still chiefly horizontal.

A strong flexure of the contour lines near streams indicates a high radial resistance, and a minor flexure a low radial resistance. As mentioned before (Section 1.3), to determine the exact shape and size of the flexures in the contour lines near streams, piezometers should be installed on both the left and right banks of the stream, and in a row perpendicular to it (Fig.6 and 21). Once the shape of the flexures in the contour lines has been determined in a few cross-sections, those of the remaining contour lines can be drawn free hand.

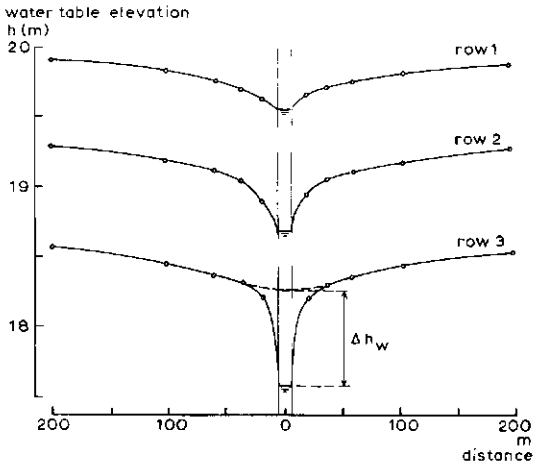


Fig.21. Water table elevation in three rows of piezometers perpendicular to a stream. The curvature in the immediate vicinity of the stream reflects the radial resistance (w) which is much greater for row 3 than for row 1.

21.3.3 THE USE OF FLOW NETS

Of the various recharge and discharge components of a groundwater system, a major problem is sometimes formed by the lateral subsurface inflow and outflow through the boundaries of the area. Half of the problem is solved, however, if one has an accurate contour map of the water table or potentiometric surface that extends somewhat beyond the limits of the area under study. To calculate the inflow and outflow through the limits of the area, one must also have information on the transmissivity of the water bearing material at these limits. Valid information on this hydraulic characteristic can only be obtained by properly conducted pumping tests (Chap.13, Vol.II).

If the limits of the area under study at the inflow and outflow sides coincide with an equipotential line and these lines are evenly spaced though different at the two limits (different but constant hydraulic gradients), the subsurface inflow and outflow rates can easily be calculated by applying Darcy's equation

$$Q = KD \frac{dh}{dx} L \quad (5)$$

where

Q = flow rate at the limit (m^3/day)

KD = transmissivity of the aquifer at the limit (m^2/day)

$\frac{dh}{dx}$ = hydraulic gradient at the limit (dimensionless)

L = width of the section through which the flow takes place (m).

Here, of course, we further assume that no flow occurs through the lateral limits of the area, in other words that these limits are flow lines.

Unfortunately the above situation is seldom found and the boundaries of the area will usually be irregularly shaped or curved. Under these circumstances equipotential lines intersect the boundaries of the area at certain, sometimes entirely different, angles. Figure 22 shows a low-lying (polder) area, surrounded by a dike which is regarded as the limit of the area. Natural streams, whose water levels are considerably higher than the land surface, occur at some distance from the dike. The contour lines show an inward flow from the streams toward the area under study.

To find the rate of this groundwater flow, we make use of a flow net, sketched along the area's limits. This flow net is constructed by drawing flow lines perpendicular to the equipotential lines in such a manner that "squares" are formed (Chap.6, Vol.I). Two adjacent flow lines may be regarded as impervious boundaries because there can be no flow across these lines. The flow rate through any channel or path of flow between two adjacent flow lines may be obtained from Darcy's equation (5). If we consider a width y of unit thickness of the aquifer measured normal to the direction of flow indicated by x , then Darcy's equation may be rewritten for this part of the aquifer as

$$\Delta q = K \frac{dh}{dx} y \quad (6)$$

where

Δq = the flow rate occurring between a pair of adjacent flow lines

y = the spacing of the flow lines.

If x represents the spacing of the equipotential lines and Δh the drop in hydraulic head between the equipotential lines, then Eq.(6) may be written as

$$\Delta q = K\Delta h(y/x) \quad (7)$$

Since the flow net was constructed to form a system of "squares", the ratio y/x is equal to unity. The same drop in hydraulic head occurs across each "square".

If there are n "squares", the total flow (q) through a unit thickness of the aquifer is expressed by

$$q = n\Delta q \tag{8}$$

As q represents the total flow through a unit thickness of the aquifer, the equation for the flow through the full thickness of the aquifer (D) becomes

$$Q = n\Delta hKD \tag{9}$$

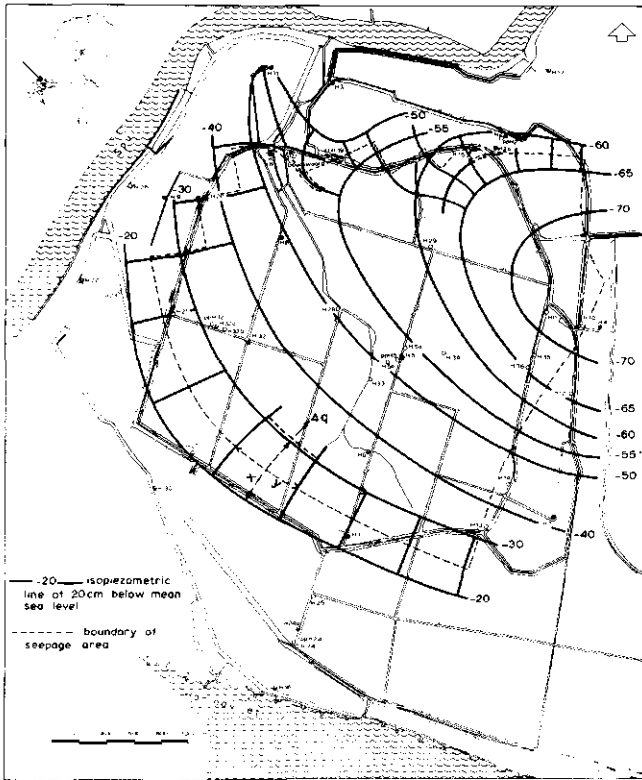


Fig.22. The use of a flow net to calculate the rate of groundwater inflow into an area at its irregularly shaped boundaries (after DE RIDDER and WIT, 1967).

From Fig.22 it can be seen that there are 16 "squares" along the boundaries of the area, of which 9 have a $\Delta h = 0.10$ m, 6 a $\Delta h = 0.05$ m, and 1 a $\Delta h = 0.025$ m. If it is assumed that $KD = 4000 \text{ m}^2/\text{day}$ it follows that

$$Q = [(9 \times 0.10) + (6 \times 0.05) + (1 \times 0.025)] \times 4000 = 4900 \text{ m}^3/\text{day}$$

If the area under study covers $692 \text{ ha} = 692 \times 10^4 \text{ m}^2$, it follows that

$$Q = 0.7 \text{ mm/day}$$

21.3.4 EVALUATION OF OTHER TYPES OF GROUNDWATER MAPS

Watertable fluctuation maps

The areas of recharge and discharge are easily detected from a watertable fluctuation map. Areas characterized by large fluctuations in water table are usually recharge areas. They generally represent topographical high-lying land with no, or only few, natural drainage channels.

Areas with minor changes in water table are usually discharge areas. They generally represent low-lying lands and topographical depressions which receive continuous inflow of groundwater from adjacent higher grounds. Because of this net subsurface inflow, the water table is high for most of the year and therefore cannot change much. A denser network of natural drainage channels may be found here than in the recharge areas.

A change in water table involves a change in the volume of groundwater stored in the soil. If the effective porosity of the zone in which the water table changes occur is known, the change in groundwater storage over a given period can be expressed as

$$\Delta S = u\Delta h \tag{10}$$

where

ΔS = change in storage of groundwater over a given period and per unit of horizontal surface area (m)

u = effective porosity of the soil (dimensionless)

Δh = change in water table elevation over the given period (m)

Hydraulic difference maps

A map showing the difference in hydraulic head between the unconfined and the semiconfined water of a groundwater system is of fundamental importance for the analysis and evaluation of the vertical flow through semipervious layers, i.e.

layers whose hydraulic conductivity is considerably lower than that of sandy aquifers.

Semiconfined aquifers are a common feature in regions suffering from high water tables. The water confined in these aquifers is often under pressure and its head may be found (considerably) above that of the unconfined water in the covering semipervious layer. This layer is thus recharged from the underlying aquifer. The flow through the semipervious layer is chiefly vertical.

To calculate the rate of this vertical upward flow of groundwater through the covering semipervious layer, Darcy's equation can be used:

$$v_z = K' \frac{h - h'}{D'} = \frac{h - h'}{c} \quad (11)$$

where

- v_z = rate of vertical flow per unit surface area (m/day)
- h = hydraulic head of the water in the semiconfined aquifer (m)
- h' = hydraulic head of the phreatic water in the covering semipervious layer (m)
- K' = hydraulic conductivity of the semipervious layer for vertical flow (m/day)
- D' = thickness of the saturated part of the covering semipervious layer (m)
- c = D'/K' = hydraulic resistance of the semipervious layer (days).

In this equation $h - h'$ represents the difference in head between the confined and the unconfined groundwater and can be read from the map. Usually this head difference is not the same everywhere in the area. It is large close to surface water bodies whose levels are above the adjacent land (Chap.13, Vol.II) and becomes smaller farther away from them. Hence it is necessary to distinguish areas that have approximately the same differences in hydraulic head.

A major problem of this method of calculation is the hydraulic resistance (c). This characteristic of the semipervious layer usually varies considerably from one site to another.

In addition to the hydraulic differences map, a hydraulic resistance map is required showing the areal distribution of c -values. This map is difficult to obtain as it requires a considerable amount of field work, including borings that fully penetrate the covering semipervious layer (to find D') and hydraulic conductivity measurements (to find K'). Under favourable conditions pumping tests may

yield values of c (Chap.12, Vol.II; Chap.25, Vol.III) but because of their high costs their number must usually be limited. When notwithstanding these drawbacks sufficient information on this characteristic is available the method can be applied (DE RIDDER and WIT, 1967).

In irrigated areas a reverse situation can often be found. Because of field irrigation losses and conveyance losses, water tables are rising. If the hydraulic head of the water in the underlying semiconfined aquifer is lower than that of the phreatic water in the covering layer, a vertical downward flow through the covering semipervious layer comes into being. The rate of this downward flow can be calculated by applying the same formula (Eq.II).

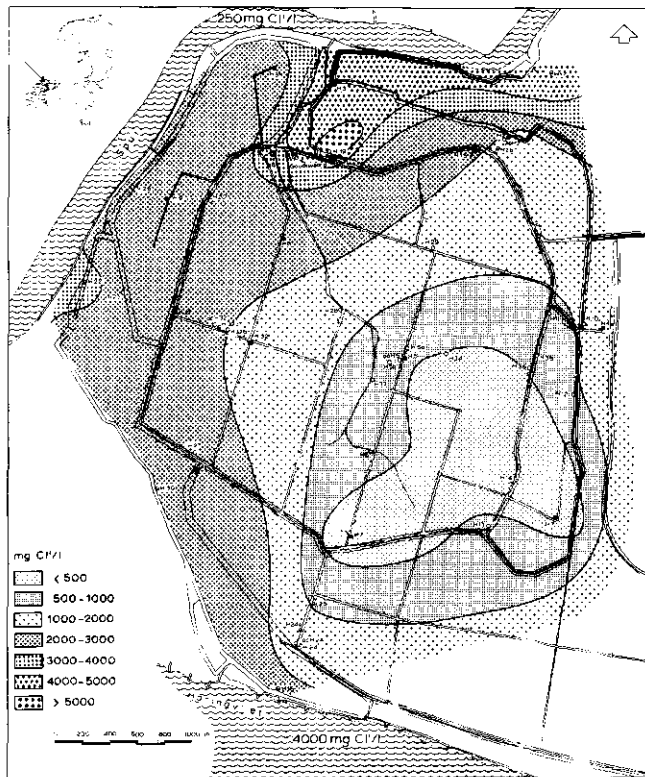


Fig.23. Chloride concentrations of the groundwater in a semiconfined aquifer at approximately 20 cm below the ground surface (after DE RIDDER and WIT, 1967).

Groundwater quality maps

Groundwater quality maps are necessary for studying soil salinity and assessing salt balances and leaching requirements (Chap.3, Vol.I; Chaps.9 and 11, Vol.II).

A groundwater salinity map can be used to convert hydraulic head data from deep piezometers that are installed in an aquifer containing groundwater of different salinity (Fig.23). The salt water heads should be converted to fresh water heads (Chap.6, Vol.I).

In areas with upward vertical flow of groundwater as mentioned in the previous section, we are interested not only in the rate of flow but also in the amounts of salt with which the upper layers and drainage channels and surface waters are charged daily. If the salt concentration of the groundwater that enters the upper layers is known, a weighted mean salt concentration can be calculated for the whole area. This value, multiplied with the mean rate of upward groundwater flow, yields the salt supply to the area (DE RIDDER and WIT, 1967).

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SURVEYS AND INVESTIGATIONS

22. ASSESSING GROUNDWATER BALANCES

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PURPOSE AND SCOPE

The assessment of a groundwater balance is of primary importance in studies on land drainage as it allows the cause(s) of the drainage problem to be determined in quantitative terms. The balance equation and its various components are discussed and its application is illustrated by some practical examples.

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22.1 INTRODUCTION

Groundwater investigations related to land drainage comprise three interrelated steps:

- an analysis of the groundwater flow system
- an analysis of the groundwater storage
- an analysis of the various components of groundwater recharge and discharge.

The objective of these investigations is to find out how to alter the groundwater flow system so as to prevent the water table from rising too far into the root zone of the crops. This might mean increasing the discharge or reducing the recharge. A common approach to enable decisions on such matters is to assess a groundwater balance of the area under study.

The water on earth - whether as water vapour in the atmosphere, as surface water in streams, lakes, and oceans, or as groundwater in the pores of the subsoil - is not at rest, but is in a continuous circulatory movement. There is a never-ending transformation from one state to another. The course of events marking the movement of a particle of water from the atmosphere to the land masses and oceans and its return to the atmosphere is known as the hydrologic cycle. The hydrologic cycle, as the word implies, has neither beginning nor end; it represents a closed system in which no single drop of water can be created or destroyed (Fig.1).

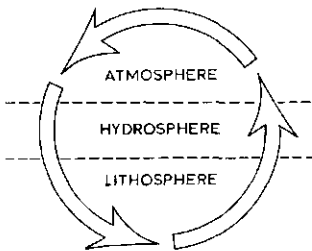


Fig.1. Schematic representation of the hydrologic cycle

In this extremely simplified representation of the hydrologic cycle we have lumped the atmosphere, the hydrosphere, and the lithosphere into single components, thus masking the fact that each component is made up of sub-components. A more informative presentation is given in Fig.2, which shows the elements of the hydrologic cycle and the way they are interrelated by the various moisture inputs and outputs. The water from the atmosphere falls to the ground as precipitation (Pr), where part of it may flow overland to streams (runoff, R.off), part will infiltrate into the soil (Inf) and percolate to the groundwater table (Perc), and part will evapo-

rate (E). Some of the precipitation does not even reach the ground surface at all; it is intercepted by the vegetation and evaporates from there. The water in the soil is partly consumed by the vegetation and returns to the atmosphere (Et). The part of the precipitation that percolates recharges the groundwater. If the water table is deep, water lost from stream channels may also recharge the groundwater supply (Q_{inf}). If the water table is shallow, water may be lost by capillary rise (Cap) and groundwater outflow to open water courses and streams (Q_{dr}). Smaller streams combine to form larger ones which carry the water to the ocean where evaporation returns it to the atmosphere and the cycle begins once more.

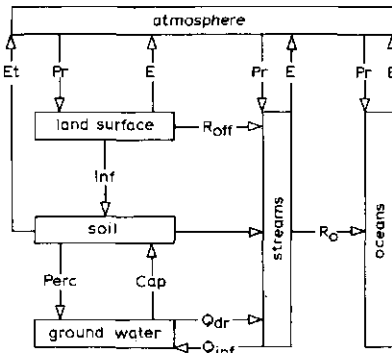


Fig.2. Elements of the hydrologic cycle

It is sometimes possible to isolate certain parts of the cycle for separate study. In land drainage, for example, we are particularly interested in the components soil and groundwater, which, taken together, may be regarded as a sub-system. But in isolating this sub-system, we cannot exclude the lines of moisture transport connecting the sub-system with the outside environment. Since sub-systems form part of a larger entity, they are open in the sense that water and energy within them are interchanged with water and energy from other components of the hydrologic cycle. A sub-system like the groundwater system is characterized by an inflow and an outflow and an amount of water stored, quantities which change with time. Continuity requires that the sub-system is in balance. In its simplest form this can be expressed by the water balance equation, sometimes referred to as the equation of hydrologic equilibrium

$$I - O = \Delta S \quad (1)$$

where I is inflow during a given period of time, O is outflow, and ΔS is the change of water in storage.

For any sub-system isolated from the hydrologic cycle, an equation of hydrologic equilibrium can be assessed. The equation can be extremely simple or more complex,

depending on circumstances. For a rain gauge, for example, it is written as

$$Pr = \Delta S \quad (2)$$

which states that the amount of precipitation (Pr) over a given period of time is equal to the measured increase of water stored in the rain gauge. The quantity ΔS in this example has a positive sign.

For a lysimeter, from which all the excess water is drained out during a certain period of time (Dr), the equation contains more members

$$Pr + I - Et - Dr = \Delta S \quad (3)$$

where

Pr = precipitation

I = water added to the lysimeter (irrigation)

Et = evapotranspiration from the lysimeter

Dr = excess water drained out from the lysimeter

ΔS = change of water stored in the lysimeter, which in this case is equal to

ΔS_{sm} = change of soil moisture in storage

The time element cannot be excluded from these equations because the individual sub-systems are open and should be treated as such by considering the inflow and outflow over a period of record. A water balance is commonly assessed for a certain period, which is chosen in such a manner that, with the exception of one, all the numerical values of the various members of the equation are known. If, for example, the period is chosen as the time between two irrigation applications, the soil moisture storage at the beginning and the end of the period is at field capacity. Hence the change of soil moisture in storage over this period is zero. Since precipitation, amount of water added, and amount of water drained are quantitatively measured, the unknown quantity of water lost by evapotranspiration can be calculated.

Care should be taken in any water balance study that all the quantities of water are expressed in the same dimensions and units; for example, in units of discharge: mm^3/day , litres/sec, etc., or in units of discharge/area: m/day, cm/day, mm/day.

In the above example the water balance equation was used to calculate one unknown quantity of the sub-system, viz. Et. It will be clear that the other quantities should be known with great precision, otherwise the result may be misleading as inaccuracies in one or more of the known quantities may exceed the magnitude of

the unknown quantity. The water balance equation can also be used as a tool to check whether all the flow components involved have been quantitatively accounted for, and what components have the greatest bearing on the problem under study. Thus, taken all round, a water balance assessment is a very useful technique to apply in any hydrologic study.

The characteristic features of the water balance can be summarized as follows:

- a water balance can be assessed for any sub-system of the hydrologic cycle, for any size of area, and for any period of time
- a water balance can serve to check whether all flow components involved have been quantitatively accounted for
- a water balance can serve to calculate the one unknown member of the equation, provided that the other members are quantitatively known with sufficient accuracy
- a water balance can be regarded as a model of the complete hydrologic process under study, and consequently can be used to predict the effect that changes imposed on certain components will have on the other components of the system or sub-system.

22.2 GROUNDWATER BALANCE EQUATION

The sub-system of soil and groundwater in which the drainage engineer is interested is shown in its most elaborate form in Fig.3.

Since the sub-system is open in the sense that water is exchanged with the outside environment, the system's boundaries and the flow components at these boundaries must be properly defined.

The situation depicted in Fig.3 shows an unconfined aquifer, also called free watertable aquifer, bounded above by the land surface and below by a clay bed whose hydraulic conductivity is low, though not zero. The clay bed in turn rests on a semiconfined aquifer. The upper, unconfined aquifer is only partly saturated, the boundary between the saturated zone and the unsaturated zone being formed by a free water table. A change in storage of water in such an aquifer is reflected by a change in the height of the water table as measured in open bore holes or observation wells. For the present purpose of land drainage, we are interested in assessing a groundwater balance for the system bounded above by the water table and below by the clay layer and furthermore by the boundaries of the area, shown as vertical lines on the left and right of the diagram. For this groundwater sub-system the following inflow and outflow components can be distinguished:

Inflow components

- Perc = percolation of water from precipitation through the unsaturated zone towards the water table (effective percolation)
- Q_{inf} = seepage from stream channels, open water courses and other surface water bodies whose water levels are higher than the watertable elevation
- Q_{up} = upward vertical flow of groundwater entering the unconfined aquifer through the underlying semipervious clay layer, which in turn covers a semiconfined aquifer with a relatively high potentiometric head
- Q_{lsi} = lateral subsurface inflow from an adjacent area with a higher water table than that in the area under consideration.

Outflow components

- Cap = capillary rise from the (shallow) groundwater table into the overlying unsaturated zone
- Q_{dr} = outflow of groundwater into stream channels, open water courses, and other surface water bodies whose water levels are lower than the watertable elevation
- Q_{do} = downward vertical flow of groundwater leaving the unconfined aquifer through the semipervious clay layer towards the underlying semi-confined aquifer with a relatively low potentiometric head
- Q_{lso} = lateral subsurface outflow into an adjacent area with a lower water table than that in the area under consideration.

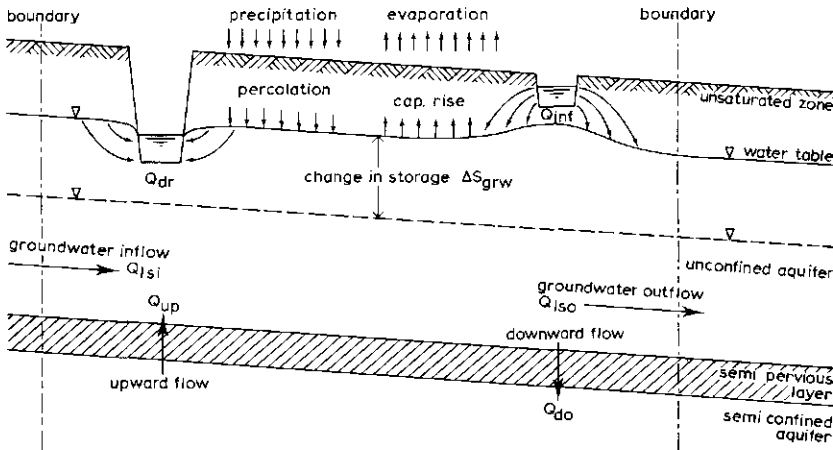


Fig.3. Flow components of a groundwater sub-system

The balance of these inflow and outflow components equals the change in groundwater storage in the unconfined aquifer. This is expressed by the symbol ΔS_{grw} . Consequently the groundwater balance equation reads

$$(\text{Perc} + Q_{up} + Q_{inf} + Q_{lsi}) - (\text{Cap} + Q_{dr} + Q_{do} + Q_{lso}) = \Delta S_{grw} \quad (4)$$

or

$$\text{recharge} - \text{discharge} = \text{change in storage}$$

A more informative presentation can be made by grouping the comparable recharge and discharge components

$$(\text{Perc} - \text{Cap}) + (Q_{inf} - Q_{dr}) + (Q_{up} - Q_{do}) + (Q_{lsi} - Q_{lso}) = \Delta S_{grw} \quad (5)$$

The above equation expresses that the changes in groundwater storage are the result of

- the recharge-discharge conditions determined by the water balance of the unsaturated zone (member: $\text{Perc} - \text{Cap}$)
- the recharge-discharge conditions determined by the position of the water level in the stream channels and open water courses relative to the water table (member: $Q_{inf} - Q_{dr}$)
- the recharge-discharge conditions determined by the potentiometric head in the underlying semiconfined aquifer (member: $Q_{up} - Q_{do}$)
- the recharge-discharge conditions determined by the lateral groundwater inflow and outflow across the boundaries of the area under consideration (member: $Q_{lsi} - Q_{lso}$).

A brief discussion of the various recharge and discharge components follows.

22.2.1 WATER BALANCE OF THE UNSATURATED ZONE (Perc-Cap)

Precipitation and irrigation (symbols: Pr. and Irr.) result in infiltration of water into the soil, either completely or partly, because some of the water may run off over the ground surface into open water courses (surface runoff; symbol R.off). The infiltrated water will be retained by the soil until the maximum soil moisture content is attained (field capacity). Any amount of water that infiltrates into the soil over and above the field capacity percolates to the water table (Perc).

On the other hand, water from the soil-moisture reservoir evaporates from the soil surface and is transpired by the vegetation cover. If this process of evapotranspiration (symbol: E_t) is in excess of precipitation and irrigation, the soil-moisture storage decreases, the moisture content becomes less than the equilibrium moisture content, and groundwater, if in capillary contact with the root zone, starts moving upward by capillary flow (Cap).

The members (Perc) and (Cap) are of great importance in any drainage study because they express the amount of water passing from the unsaturated zone (root zone) to the saturated zone and vice versa.

The water balance of the unsaturated zone can be written as follows (Fig.4)

$$(Pr + Irr - R.off) - Et - (Perc - Cap) = \Delta S_{sm} \quad (6)$$

where ΔS_{sm} stands for the change in soil-moisture storage.

Soil-moisture storage is here regarded as the amount of water retained by the soil up to a maximum soil-moisture content equal to field capacity. Near the water table, in the capillary fringe, water is also retained at higher moisture contents, but it is assumed that this amount of water remains constant throughout the balance period. Thus, when the soil-moisture content is at field capacity at the beginning and at the end of the considered period, ΔS_{sm} will be zero, irrespective of any change in the water table height.

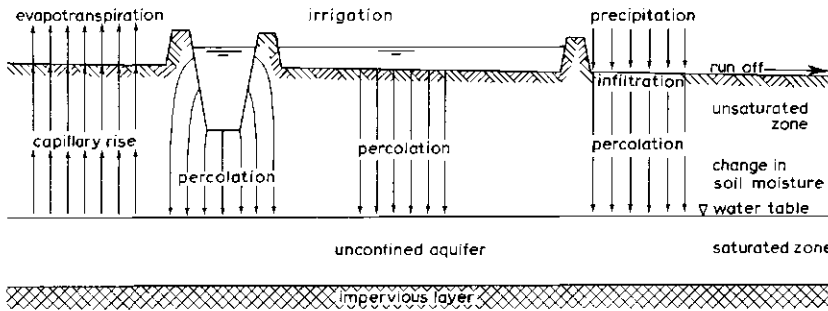


Fig.4. Schematic presentation of the flow components in the unsaturated zone

22.2.2 WATER BALANCE OF SURFACE-WATER SYSTEM ($Q_{inf} - Q_{dr}$)

Stream channels, open water courses, and other surface-water bodies such as lakes, influence the height of the water table to some extent. When the water level of these water courses is higher than the water table elevation, water is lost to the underground and the groundwater is thus recharged (Q_{inf}). If, on the other hand,

the level of the stream channels is lower than the water table elevation, ground-water flows out into these channels (drainage) and is discharged by them (Q_{dr}).

The inflow and outflow of surface water, can be determined by stream flow measurements, and the resulting balance will show the quantity that enters the surface-water system minus the quantity that is lost. The water balance for the surface-water system can be written as follows

$$(\text{Inf} - \text{Outf}) - (Q_{inf} - Q_{dr}) = \Delta S_{sw} \quad (7)$$

where ΔS_{sw} stands for the change in surface-water storage. This quantity can usually be neglected if the water level of the channel network remains approximately the same during the considered time period. Note that for extensive bodies of surface water and for long time periods evaporation from these open water surfaces cannot be neglected.

22.2.3 INFLUENCE OF UNDERLYING SEMICONFINED AQUIFER ($Q_{up} - Q_{do}$)

If an unconfined aquifer is underlain by a clay layer whose hydraulic conductivity for vertical flow is low, though not zero, and if this clay layer in turn rests on an aquifer containing water under pressure, substantial quantities of water may enter the unconfined aquifer by upward vertical flow through the clay bed (Q_{up}). This occurs when the potentiometric head of the confined water in the deep aquifer is higher than that of the unconfined water in the upper aquifer. In the reverse situation the upper aquifer may lose water to the deep aquifer, leakage (Q_{do}).

The rate of this upward or downward vertical flow through the clay layer (v_z) can only be determined by direct methods if the hydraulic resistance (c) of the clay bed and the potentiometric heads of the water in the upper and lower aquifers (h_1 and h_2) are known. The flow rate can then be found by applying Darcy's equation

$$v_z = K' \frac{h_2 - h_1}{D'} = \frac{\Delta h}{c} \quad (8)$$

where K' is the hydraulic conductivity of the clay bed for vertical flow, D' is the thickness of the clay bed, and $D'/K' = c$ is the hydraulic resistance of the clay bed.

In relatively flat areas under natural conditions the two vertical flow components, Q_{up} and Q_{do} , seldom occur simultaneously. If the semiconfined aquifer is in direct contact with a deeply incised river whose water level changes appreciably during the course of the year, the flow components Q_{up} and Q_{do} may appear alternately

(Chap.13, Vol.II). Under these circumstances the recharge and discharge by vertical movement of groundwater may constitute an important, though not constant, factor in the groundwater balance.

Determining these flow components by direct measurements is usually a rather costly affair, because a network of double piezometers and quite a lot of fieldwork, including pumping tests, is required, to find the values of the hydraulic resistance, c . Hence it is common practice to derive the order of magnitude of these flow components indirectly from the groundwater balance. A prerequisite is that the values of the other members of the balance equation are known with sufficient accuracy.

22.2.4 INFLUENCE OF LATERAL GROUNDWATER FLOW ($Q_{lsi} - Q_{lso}$)

The boundaries of the area under study usually do not represent streamlines, i.e. they are not perpendicular to the equipotential lines. Hence the lateral flow of groundwater crossing the area's boundaries must be accounted for in the groundwater balance equation.

If the streams in the area under study are draining (Q_{dr}) and none of them is losing water to the underground ($Q_{inf} = 0$), and if the clay bed underlying the unconfined aquifer is impervious (Q_{up} and Q_{do} are zero), and, moreover, if only periods of long duration are considered (change in groundwater storage, ΔS_{grw} is very small), the groundwater balance equation (5) can be replaced by (see Fig.5)

$$(Q_{lsi} - Q_{lso}) = Q_{dr} - (\text{Perc} - \text{Cap}) \quad (9)$$

This equation shows how important it is to know the quantity ($Q_{lsi} - Q_{lso}$), because it represents the difference between the average quantity of water that has to be drained and the average supply ($\text{Perc} - \text{Cap}$).

For the difference between the lateral subsurface inflow and outflow the symbol I_{ss} is used, hence

$$I_{ss} = Q_{lsi} - Q_{lso} \quad (10)$$

Sometimes the term "seepage" is used for the symbol I_{ss} , but, since this word has different meanings, ERNST, DE RIDDER, and DE VRIES (1970) introduced the term "net subsurface inflow" as an addition to hydrologic terminology, thus replacing various other, often confusing, terms currently used. Obviously the net subsurface inflow (I_{ss}) may attain positive or negative values, depending on differences in water level (and also in ground surface elevation) between the area under consideration and the surrounding area.

The practical consequences of the value and sign of I_{ss} are of great importance in the analysis of the hydrological conditions prevailing in the area. A negative net subsurface inflow ($-I_{ss}$) indicates that the supply exceeds the outflow of groundwater into streams and other open water courses: $(Perc - Cap) > Q_{dr}$. This means that there is no danger of cumulative salinization of the root zone. This situation may be found in areas whose land surface and water table elevation are higher than those of the surrounding area. A positive net surface inflow ($+I_{ss}$) indicates that the supply is less than the outflow of groundwater into streams and other open water courses: $(Perc - Cap) < Q_{dr}$. This means that the area is receiving "foreign" water from outside. As a consequence, the water table may rise to heights very near to, or even at, the ground surface, and artificial drainage will be required to control the water table and to prevent the root zone from cumulative salinization. This situation is often found in topographical depressions and in low-lying valley bottoms.

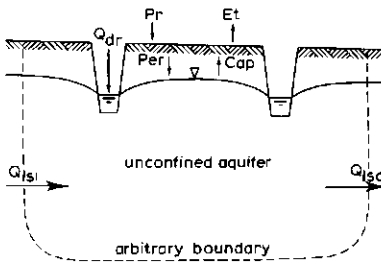


Fig.5. Unconfined aquifer with flow components contained in Eq.9

If the water table gradient and the transmissivity of the aquifer are known, the lateral groundwater inflow and outflow across the project area's boundaries can be calculated directly, either by using Darcy's equation or a flow net analysis (Chap.2), Vol.III). To determine the aquifer's transmissivity at the boundaries, it will be necessary to perform several pumping tests which, because of their high costs, will usually be limited in number. Without sufficient and reliable data on the transmissivity, it is impossible to make a direct calculation of the inflow and outflow across the boundaries. Recourse can then be made to indirect methods and the net subsurface inflow (I_{ss}) can be derived from the groundwater balance, provided the values of the other members of the equation are known with sufficient accuracy.

Alluvial plains are seldom entirely flat, but instead show often minor relief features in the form of topographical ridges, mounds, depressions, etc. In the special case that a groundwater mound (an area which is surrounded at all sides by

a lower water table) is found in the area under consideration, the boundary of the area can be chosen in such a manner that it coincides with the boundary of the groundwater mound. The area is then an area without any groundwater inflow ($Q_{lsi} = 0$), and the groundwater outflow from the area is equal to the net subsurface inflow ($Q_{lso} = -I_{ss}$, see Eq.10). If no stream channels occur in the area ($Q_{dr} = 0$), the lateral groundwater outflow will equal the supply ($Q_{lso} = \text{Perc} - \text{Cap}$). Similarly, for an area whose boundaries coincide with those of a depression in the water table (an area which is surrounded on all sides by a higher water table), the groundwater outflow is zero ($Q_{lso} = 0$). Hence the groundwater inflow across the boundaries equals the net subsurface inflow which, in this case, has a positive sign ($Q_{lsi} = +I_{ss}$).

22.2.5 CHANGE IN GROUNDWATER STORAGE (ΔS_{grw})

As a result of the hydrologic cycle, the quantity of groundwater stored in the unconfined aquifer under discussion constantly increases by inflow of water from various sources, as discussed previously. At the same time the aquifer loses water by various types of outflow. In the long run, however, inflow equals outflow and a near-stationary water table occurs, with seasonal fluctuations around the average level.

Hence, if a groundwater balance is to be assessed for a relatively short period, the inflow may not balance the outflow, the difference being accounted for as the change in groundwater storage. A change in storage of groundwater is reflected by a rise or fall of the water table (symbol: Δh).

One of the factors influencing the change in water table is the effective porosity (drainable pore space) of the soil zone in which the water table fluctuations occur (symbol: μ). Since BOUSSINESQ (1903) pointed out that formulas from the theory of heat conduction in solids can also be used for solving problems of unsteady flow of groundwater, the following equation has been widely used in groundwater hydrology (see also Chap.6, Section 5.4, Vol.I)

$$KD \left(\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} \right) = \mu \frac{\partial h}{\partial t} - R \quad (11)$$

where

- h = potentiometric head of groundwater in horizontal flow
- x,y = horizontal coordinates
- t = time
- K = hydraulic conductivity of homogeneous aquifer

- D = thickness of aquifer
- KD = transmissivity of aquifer
- R = effective recharge through upper boundary of the considered area
- μ = effective porosity.

From this differential equation it follows immediately that the effective porosity is defined as the ratio of the change in storage of groundwater during a certain period to the corresponding change in potentiometric head, or

$$\mu = \frac{\Delta S_{grw}}{\partial h / \partial t} \quad (12)$$

The effective porosity is often also defined as the percentage of the total volume of soil occupied by the ultimate volume of water released from or added to storage in an unconfined aquifer per unit horizontal area of aquifer and per unit decline or rise of the water table. The change in groundwater storage over a certain period and per unit horizontal area is given by

$$\Delta S_{grw} = \mu \Delta h \quad (13)$$

Frequent recordings of the groundwater table taken from a representative network of observation wells can be used to draw hydrographs from which the value of Δh can be read for any chosen period.

Equation 13 might suggest that, like hydraulic conductivity, the effective porosity is a hydrologic constant. This is not entirely true. Firstly, a lowering of the water table is not accompanied by an instantaneous gravity drainage of the soil pores. The amount of water drained from the soil increases, though at a diminishing rate, as the time of drainage increases. Hence the effective porosity increases as the time of drainage by gravity increases. JOHNSON (1966) reports that a pumping test performed near Grand Island (Nebr.) drained the sand and gravel in such a manner that the computed effective porosity of the material was 9,2 % after 6 hours of pumping, 11,7 % after 12 hours, 16,1 % after 24 hours, 18,5 % after 36 hours, and 20,1 % after 48 hours of pumping. Consequently the values of the effective porosity of an unconfined aquifer derived from pumping tests data should be considered with great caution unless pumping is continued for a sufficiently long time.

Secondly, it has been recognized that the effective porosity changes as the depth to water table changes. A change in water table elevation is accompanied by a change in soil moisture content of the entire soil profile, both in the zone in which the water table fluctuations occur and above this zone. This means that,

in general, the effective porosity of the entire profile decreases as the water table rises. Table 1 gives an example of this phenomenon for two different soil types with an initial water table at 1 m below land surface.

Table 1. Effective porosities of a soil profile with rising water table

Total rise in water table (cm)	Effective porosity of	
	marine clay	loamy sand
30	5,4%	22,3%
50	5,1%	19,5%
70	4,9%	16,8%
100	4,3%	13,0%

Hence, whenever the effective porosity is being considered, the depth of the initial water table should be taken into account. For a further discussion on the subject the reader is referred to Chap.20, Vol.III and to VAN HOORN (1960) and ERNST (1969).

The effective porosity can be derived from soil moisture characteristics (pF-curves: see Chap.20, Vol.III) and from the (known) amount of water released from or added to storage and the corresponding change in water table elevation; a prerequisite is that the moisture content before and after the water-table change corresponds with the equilibrium moisture content. The equilibrium moisture content is the moisture content at which the suction corresponds with the height above the water table (VAN HOORN, 1960). It is recommended that measurements be taken in a period during which the moisture deficit of the soil profile and the evaporation are so slight that they can be neglected. During this period the moisture content will correspond as closely as possible with the equilibrium moisture content (Chap.26, Vol.III).

It should furthermore be noted that if the water table drops, part of the water is retained by the soil particles; if it rises due to rainfall air can be trapped in the interstices that are filling with water. Hence the effective porosity for rising water is, in general, less than that for a falling water table.

The effective porosity varies with grain size and sorting of the sediments. The lowest values are found for clay and clayey materials (less than $\frac{1}{2}$ % to 5%) and the highest for coarse sand (20 to 35%). For uniform sediments (well-sorted) the effective porosity is higher than for non-uniform sediments (poorly sorted). One might expect gravels to have the highest effective porosities, but, due to their low degree of sorting, such materials usually show a rather wide range of effective porosity values. Effective porosity ranges for different materials, as reported by JOHNSON (1966), are shown in Table 2.

Table 2. Effective porosity ranges for different materials
(after JOHNSON, 1966)

Material	Effective porosity %	
	Range	Mean
Clay	0 - 5	2
Silt	3 - 19	8
Sandy clay	3 - 12	7
Fine sand	10 - 32	21
Medium sand	15 - 32	26
Coarse sand	20 - 35	27
Gravelly sand	20 - 35	25
Fine gravel	17 - 35	25
Medium gravel	13 - 26	23
Coarse gravel	12 - 26	22

22.3 SOLUTION OF THE GROUNDWATER BALANCE EQUATION

If the groundwater balance equation is to be applied effectively, it is essential that both the area and the time period for which the balance is assessed be carefully chosen. In making the right choice recourse should be made to all available information on precipitation pattern, irrigation practices, evaporation data, etc. Well hydrographs will also be of great value.

A comparison of the well hydrographs allows wells with similar watertable fluctuation patterns to be grouped. In this manner sub-areas with a homogeneous groundwater regime can be distinguished, i.e. the water table in each part of the sub-area reacts similarly to the prevailing recharge and discharge conditions (Fig.6). The groundwater balance is then assessed for these sub-areas, or for a single sub-area, as is required. It is also conceivable that the whole area be divided into sub-areas by the Thiessen method. The watertable changes observed in a single well are then considered to be representative of the sub-area surrounding that well.

Well hydrographs can also be used in selecting the proper time period. A period can be chosen during which there is a general rise or fall of the water table, or a period at the beginning and end of which the water table elevation is the same, so there is no change in storage of groundwater (Fig.7). An examination of the recharge and discharge conditions allows a time period to be selected which is characterized, for example, by known quantities, or by the absence of precipitation and irrigation.

The principle of this approach is that the area and the time period are chosen in such a manner that the conditions of recharge, discharge, and storage are

uniform and quantitatively known, except for one flow component. This component can then be solved from the equation. The component thus found can subsequently be introduced into the equation for another chosen time period with another unknown item, provided the value of the component remains reasonably constant. In general, lateral groundwater flows remain rather constant for different time periods, whereas percolation and capillary rise usually vary during different parts of the hydrological year. The calculations should preferably be made for more than one time period in order to have a check on the results obtained. It is not always possible, nor necessary, to solve all the individual members of the groundwater balance equation separately. Sometimes, depending on the problem under study, a number of members can be lumped, and their net value only be taken.

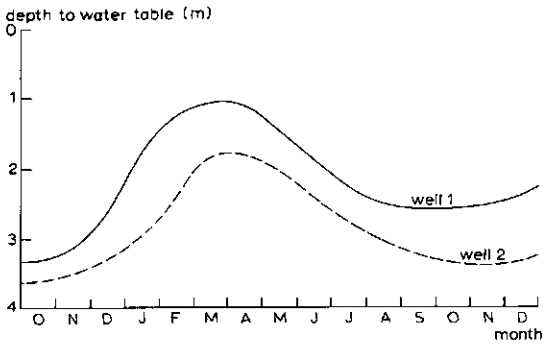


Fig.6. Similarly reacting well hydrographs

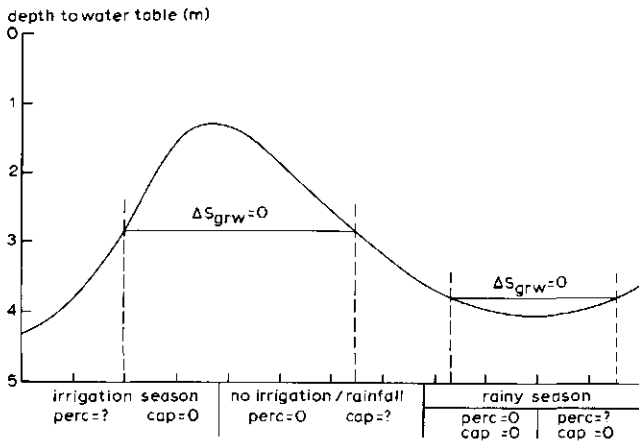


Fig.7. The use of a well hydrograph to select convenient time periods for assessing groundwater balances

The most important flow components for land drainage are

- percolation and capillary rise (contact between saturated and unsaturated zone),
- net subsurface inflow (groundwater flow components),
- outflow of groundwater into streams and open water courses, and water losses from these water courses to the underground (contact between surface water and groundwater),
- change in storage of groundwater.

Suggestions for their quantitative determination are given in the following sections.

22.3.1 PERCOLATION (Perc) AND CAPILLARY RISE (Cap)

Percolation and capillary rise are two flow components that are of primary importance in analyzing the critical conditions of groundwater recharge and the salt balance of the root zone. Theoretically speaking, the two components do not occur at the same time, but during fairly long periods, for instance an irrigation season, they may appear alternately: percolation while irrigation water is being applied and for 2 to 5 days afterwards; capillary rise during the remaining days until the next irrigation. In the quantitative determination of these flow components, the following situations might be encountered:

a) During the rainy season, precipitation (measured) exceeds potential evapotranspiration (calculated) and the soil moisture content remains at field capacity. Under these circumstances capillary rise will be zero and percolation equals precipitation minus evaporation and, if applicable, minus runoff. Hence,

$$\begin{aligned} \text{Perc} &= \text{Pr} - \text{Et} \\ \text{and Cap} &= 0 \end{aligned} \tag{14}$$

b) The same as under a) but, at the beginning of the rainy season, the soil moisture content is not yet at field capacity. Precipitation in excess of evaporation causes only an increase in storage of soil moisture. Percolation does not yet occur and capillary rise has become so small that it can be neglected. Hence,

$$\begin{aligned} \text{Pr} - \text{Et} &= \Delta S_{sm} \\ \text{and Perc} &= \text{Cap} = 0 \end{aligned} \tag{15}$$

c) The same as under a) but the considered period is short, covering only a few days of heavy rainfall, during which the soil moisture content is at field

capacity. Evaporation during this short and wet period can be neglected. The total rainfall, after reduction for runoff, may be accounted for as percolation. Hence,

$$\begin{aligned} \text{Perc} &= \text{Pr} \\ \text{and} \quad \text{Cap} &= 0 \end{aligned} \quad (16)$$

d) During the irrigation season percolation occurs due to over-irrigation, i.e. irrigation is in excess of the available soil moisture storage. From irrigation flow measurements, or data on the irrigation efficiency, the irrigation losses can be derived which, in this situation, equal percolation. Hence,

$$\text{Perc} = \text{Irr} - \Delta S_{sm} \quad (17)$$

If the chosen time period covers only a few days during and after irrigation, capillary rise can be assumed negligible. For longer periods, capillary rise may attain considerable values that are not easy to determine by direct methods. In this situation capillary rise can be derived from the water balance equation itself (see Section 2.1). Hence,

$$\text{Cap} = 0 \quad \text{or} \quad \text{Cap} = ? \quad (18)$$

e) For periods during which the water table is at a depth of about 4 m or more below the ground surface, the capillary rise decreases to negligibly low values, irrespective of the moisture conditions in the unsaturated zone. Hence,

$$\text{Cap} = 0 \quad (19)$$

During the dry season, when the soil moisture storage is partly depleted and the water table is at less than, say, 3 m below ground surface, capillary rise will not be zero but instead may attain large values, depending on the type of soil (Chap.II, Vol.II).

If the groundwater quality is good and sufficient leaching is provided during irrigation applications or during the wet season, capillary rise may be considered a favourable factor, because it represents a supplemental supply of water to the crops.

If, on the other hand, the groundwater is saline and percolation is limited during the wet season, capillary rise will constitute the principal factor in the process of gradual salinization of the root zone.

Capillary rise is difficult to determine directly, and then only on the basis of experimental field research. However, once sufficiently accurate values of the effective porosity and the net subsurface inflow have been obtained, it can be

computed from the groundwater balance equation which, in this case, is written as

$$\text{Cap} = \text{Perc} + I_{ss} - \mu\Delta h \quad (20)$$

22.3.2 NET SUBSURFACE INFLOW (I_{ss})

The net subsurface inflow has been defined as the difference between the lateral groundwater inflow and outflow (Eq.10). Under natural conditions many drainage problems are caused by this flow component which, in such cases, has a positive value ($Q_{lsi} > Q_{lso}$). The amount of inflow in excess of outflow causes the water table to rise.

Upward flow of groundwater through a semipervious clay bed that is found at some depth below the area under study may also contribute to the drainage problem. In this case the net value of ($Q_{up} - Q_{do}$) has a positive sign. If a negative net value is found, i.e. if Q_{do} exceeds Q_{up} , water from the upper unconfined aquifer leaks through the clay bed to the underlying aquifer. Because of this leakage the net groundwater inflow into the upper unconfined aquifer is reduced and this may, in a favourable sense, affect the drainage problem.

As mentioned previously, it is only by intensive and rather costly investigations that the individual flow components can be determined. As a first approximation and under certain, well-specified conditions, the total net effect of the individual groundwater flow components can be derived from the groundwater balance. If we lump these components, we may express this total effect (I'_{ss}) by the following equation

$$I'_{ss} = (Q_{lsi} - Q_{lso}) + (Q_{up} - Q_{do}) \quad (21)$$

To illustrate how the value of I'_{ss} is obtained, let us choose a period during which the channel network is dry so that the flow components Q_{inf} and Q_{dr} are zero. The groundwater balance equation then reads

$$(\text{Perc} - \text{Cap}) + I'_{ss} = \Delta S_{grw} = \mu\Delta h \quad (22)$$

Under the following conditions the quantity I'_{ss} can be determined.

a) For a period during which the change in water table is zero or so small that it can be neglected ($\Delta h = 0$, so $\Delta S_{grw} = 0$) and for which the supply ($\text{Perc} - \text{Cap}$) is known, the net value of the various leakage and lateral flow components equals the supply. Hence,

$$I'_{ss} = (\text{Cap} - \text{Perc}) \quad (23)$$

b) If the water table is changing during the chosen period, and if the supply and the effective porosity are known, the net value of the leakage and lateral flow components is given by

$$I'_{ss} = \mu \Delta h - (\text{Perc} - \text{Cap}) \quad (24)$$

If both percolation and capillary rise are zero (no rainfall or irrigation and a deep water table) the above equation reduces to

$$I'_{ss} = \mu \Delta h \quad (25)$$

c) If the effective porosity of the soil layer in which the changes in water table occur is not yet known, both the net value of I'_{ss} and the effective porosity can be calculated from two different periods for which the supply (Perc - Cap) and the change in water table (Δh) are known. The solution implies that both I'_{ss} and μ are assumed to be constant, a condition which is satisfied if the water table during the two periods is at the same elevation. The equations read

$$I'_{ss} = \mu \frac{\Delta h_1}{\Delta t_1} - \frac{(\text{Perc} - \text{Cap})_1}{\Delta t_1} \quad (26)$$

$$I'_{ss} = \mu \frac{\Delta h_2}{\Delta t_2} - \frac{(\text{Perc} - \text{Cap})_2}{\Delta t_2}$$

22.3.3 GROUNDWATER OUTFLOW INTO STREAMS AND WATER LOSSES THROUGH STREAM BEDS (Q_{dr}) AND (Q_{inf})

Surface water and groundwater are closely related, irrespective of the moisture conditions in the unsaturated zone. If there is a surface water system (streams, canals, ditches, lakes) within the boundaries of the area under study, the outflow of groundwater to, or the water losses from, the open water courses can be determined from a water balance of the surface water system. For this purpose stream flow measurements should be taken at properly selected sites.

If only a few small stream channels occur inside the considered area, water losses from, or outflow of groundwater to, the channels may be so small that they can be neglected.

In irrigation areas the entire surface-water system may consist of irrigation canals. If there are no drainage channels, so that $Q_{dr} = 0$, the water losses to the underground will equal the conveyance losses of the irrigation system, corrected for evaporation, if need be.

22.3.4 CHANGE IN GROUNDWATER STORAGE (ΔS_{grw})

As follows from Eq.13, the change in groundwater storage over a certain period can be calculated if the effective porosity and the change in water table are known. Water table changes can easily be measured in open bore holes or observation wells, but it is difficult to determine the effective porosity directly. Sufficiently accurate values of the effective porosity can, however, be derived from the groundwater balance equation provided that the recharge, discharge, and change in water table are known precisely. These conditions are usually satisfied in experimental fields (Chap.26, Vol.III). If the discharge of such an experimental field is not measured directly, the time period(s) should be chosen in such a manner that the discharge components can be assumed to be negligible compared with the recharge components.

In determining the effective porosity, the following situations might be encountered:

a) Short periods with a high rate of known percolation following intense rainfall or irrigation. For such a short period the recharge or discharge resulting from groundwater flow (lateral inflow and outflow) can be neglected. If the change in water table during this period is measured, the effective porosity can easily be found from

$$\mu = \text{Perc}/\Delta h \quad (27)$$

b) If the outflow of groundwater into open water courses (Q_{dr}) has already been determined during a period in which the water table did not change, longer periods of irrigation or rainfall may also be used to calculate the effective porosity. The formula to be used in this case is

$$\mu = (\text{Perc} - Q_{dr})/\Delta h \quad (28)$$

22.4 EXAMPLES

The discussion of the groundwater balance will now be illustrated by some examples.

Example 1

Underneath an area of 200 ha is a clay bed 4 m below the ground surface. This bed is 1.20 m thick and its hydraulic conductivity for vertical flow $K' = 5$ mm/day. During a period of 20 days the water table in the sandy layer above the clay bed is at an average depth of 2.80 m below the ground surface. The potentiometric surface of the water confined in a sand layer below the clay bed is 1 m below the ground surface. Calculate the quantity of water that flows during this period from the deep semiconfined aquifer through the clay bed into the upper unconfined aquifer.

Groundwater balances

The hydraulic resistance of the clay bed $c = D'/K'$ or $1.20/0.005 = 240$ days. The head difference $h_2 - h_1 = 2.80 - 1.00 = 1.80$ m. Substituting these values into Eq.8 gives

$$v_z = 1.80/240 = 0.0075 \text{ m/day} = 7.5 \text{ mm/day}$$

For a period of 20 days, $v_z = 20 \times 0.0075 = 0.150$ m. For an area of 200 ha ($2 \times 10^6 \text{ m}^2$) this corresponds to

$$Q_{\text{up}} = 2 \times 10^6 \times 0.150 = 300\,000 \text{ m}^3.$$

Example 2

A rainshower of 40 mm falls on an agricultural field that has just been irrigated. It causes the water table to rise 40 cm. What is the effective porosity of the soil in which the rise in water table occurs?

After irrigation the soil profile is at field capacity and cannot store any additional water. Hence the entire 40 mm of rain will percolate to the water table. The effective porosity is found from Eq.27

$$\mu = 40/400 = 0.10 \quad \text{or} \quad 10 \text{ percent.}$$

Example 3

The same field is irrigated at intervals of 10 days. Each time an irrigation application of 60 mm is supplied bringing the soil to field capacity. Of each irrigation application 10% is lost by surface runoff. After the first irrigation the water table was at 2.00 m below ground surface. After the fourth irrigation the water table is at a depth of 1.80 m below ground surface.

During this irrigation period of 30 days, there is a rainfall of 45 mm from which no surface runoff was observed. There is an impervious layer at a depth of 4.00 m below ground surface, while lateral inflow and outflow through the field boundaries during the period of irrigation can be neglected.

Calculate now with these data the evapotranspiration E_t .

Since the soil profile is at field capacity after the first and the fourth irrigation there is no change in storage of soil moisture ($\Delta S_{\text{sm}} = 0$). The total amount of irrigation water applied during this period equals 3×60 mm (second, third and fourth irrigation application). From this amount (180 mm), 10% or 18 mm is lost by surface runoff and the remaining $180 - 18 = 162$ mm penetrates into the soil, as does the rainfall of 45 mm. The effective porosity being 10%, the 20 cm rise in water table represents an amount of

$$(200 - 180) \times 0.1 = 2 \text{ cm} \quad \text{or} \quad 20 \text{ mm}$$

Substituting the above values in Eq.6 gives

$$45 + 162 - E_t - 20 = 0$$

which gives

$$E_t = 187 \text{ mm} \quad \text{or} \quad 187/30 = 6.2 \text{ mm/day}$$

Example 4

The following data covering a 30-day period are available: percolation = 50 mm, rise in water table $\Delta h = 60$ cm. The water table is at such a depth that the capillary rise can be neglected (Cap = 0).

For another period, lasting 20 days, a percolation of 20 mm and a rise in water table $\Delta h = 30$ cm is observed. The water table is still at a depth of more than 3 m, so that the capillary rise can be neglected. Calculate the rate of net subsurface inflow into the area and the effective porosity.

On substitution of the above values into Eq.26 we obtain

$$I'_{ss} \times 30 = 600 \mu - (50 - 0)$$

$$I'_{ss} \times 20 = 300 \mu - (20 - 0)$$

After rearranging and solving for I'_{ss} we find

$$30 I'_{ss} = 600 \mu - 50$$

$$40 I'_{ss} = 600 \mu - 40$$

$$\hline 10 I'_{ss} = 10$$

$$I'_{ss} = 1 \text{ mm/day}$$

Substitution of this value into the equation for the first period, gives

$$30 \times 1 = 600 \mu - 50$$

$$600 \mu = 80$$

$$\mu = 0.13$$

Note: It will be seen that relatively slight variations in the value of the percolation of the second period have a great effect on the results of the calculations and may even change the net subsurface inflow into negative values. Hence it is preferable to choose more than two time periods, which could even be taken from different years, and to treat them graphically to smooth out inaccuracies of the individual data.

Example 5

An agricultural area is irrigated at 12-day intervals. It is known from irrigation investigations that during each application approximately 25 mm of irrigation water is lost to the water table (the sum total of field application losses and seepage losses from the canal system). The lateral groundwater inflow through the boundaries of the field exceeds the lateral outflow by an amount of about 0.5 mm/day (positive net subsurface inflow $I_{ss} = 0.5$ mm/day). The effective porosity of the soil is 8%. During a period of 36 days (including three irrigation applications) a 50 cm rise in water table is observed. Calculate the capillary rise.

Substituting the above values into Eq.26, gives

$$\text{Cap} = (3 \times 25) + (36 \times 0.5) - (0.08 \times 500) = 53 \text{ mm}/36 \text{ days}$$

or an average rate of capillary rise of 1.5 mm/day.

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SURVEYS AND INVESTIGATIONS

23. MEASURING SOIL MOISTURE

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PURPOSE AND SCOPE

A discussion of methods of determining soil moisture tensions and soil moisture contents, with comments on some soil moisture characteristics.

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23.1 SOIL MOISTURE RETENTION

23.1.1 INTRODUCTION

Between the soil and its water there exists a continuous energy relationship, the theoretical background of which has been dealt with in Chap.5, Vol.I.

The potential energy of the soil water is defined as the work that is required to bring a unit mass of water from a given reference position to a particular point in the soil. Usually designated as reference position is the phreatic surface or the groundwater table. In the zone above the groundwater table the potential - which is numerically equal to the hydrostatic pressure - has a negative sign. Since the potential is negative in the unsaturated zone it is convenient to express it in terms of tension or suction (negative pressure) involving only a change in algebraic sign. In an equilibrium situation the tension in a soil profile above a water table equals the height above that water table.

The mutual relationship between soil moisture content and soil moisture tension is to a considerable extent dependent on the size and geometrical arrangement of the pores in the solid matrix of the soil.

Cohesion forces (forces of attraction between molecules of the same type) and adhesion forces (forces of attraction between molecules of different type) together constitute matric forces or capillary forces. Combined with the "osmotic" forces exerted by the ions adsorbed to the solid phase, they form the binding forces between solid and liquid phase in a soil. The matric forces can be referred to as soil moisture tension, soil moisture suction, or matric suction. The dissolved solutes in the soil water may cause an osmotic solute tension or suction. The sum of soil moisture tension and solute tension is called total soil moisture tension.

In sandy soils the soil moisture tension is mainly built up by capillary forces whereas in clay soils, particularly in the lower water content range, the adsorbed ions play an important part.

23.1.2 RELATION TO PORE SIZE

The capillary forces induce a concave water meniscus at the water-air interface. Under hydrostatic equilibrium conditions (no water flow) the height of capillary rise in soil pores, which are assumed to have a cylindrical shape, can be calculated with the following equation (Fig.1)

$$\sigma \cos \alpha 2\pi r = \pi r^2 \rho_w g h \quad (1)$$

where

- σ = surface tension of water against air (dynes/cm)
- α = contact angle between water and wall of capillary
- r = radius of capillary (cm)
- g = acceleration due to gravity (cm/sec² or dynes/gram)
- h = length of water column above a free water level (cm)
- ρ_w = density of water (gram/cm³)

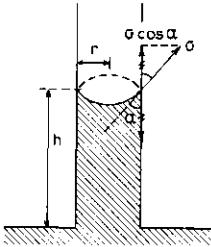


Fig.1. Capillary rise of water

Lifting force = vertical component of surface tension (σ) acting on the internal circumference of the capillary ($2\pi r$)

Downward force = weight of water column = mass of water column $\times g$ = volume of water column \times density $\times g$ = $\pi r^2 h \rho_w g$

Under equilibrium conditions $\sigma \cos \alpha 2\pi r = \pi r^2 h \rho_w g$

Since the soil matrix is strongly hydrophillic, the contact angle α of liquid water and soil solids - perhaps with the exception of dry organic soils - will tend towards zero ($\cos \alpha = 1$). Since the density of water can be assumed to equal 1, and the surface tension σ to equal 72.75 at 20 °C (Table 1), Eq.1 reduces to

$$h = \frac{2 \times 72.75}{981 \times r} = \frac{0.30}{d} \quad (2)$$

where d = equivalent diameter of pore (cm).

It can be concluded from Eq.2 that the height of capillary rise is inversely proportional to the diameter of the pore.

Table 1. Dependence of surface tension of water against air (σ) on temperature (T)

T (°C)	σ (dynes/cm)
0	75.60
10	74.22
15	73.50
20	72.75
25	71.97
30	71.18

Soil moisture

Since the soil moisture tension (S_m) can be expressed as the height of a water column (h) above the free water surface, it follows from Eq.2 that with decreasing pore diameter the soil moisture tension will increase, and vice versa. Consequently in the drying process a soil will first release water from the larger pores; the smaller the pores the greater the tension force needed to empty them. Since the soil moisture tension range extends from 0 (saturated soil) to about 10^6 cm water column (air-dry soil), a logarithmic scale is often used in which the symbol pF equals \log (cm water). In that case Eq.2 becomes

$$pF = \log h = \log S_m = - 0.523 - \log d \quad (3)$$

This relation is presented in Fig.2; some values are given in Table 2.

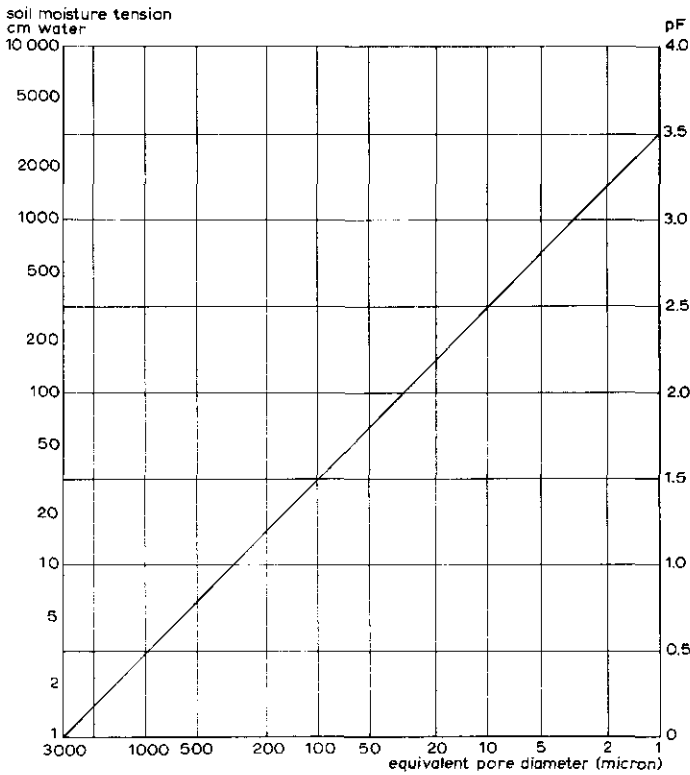


Fig.2. Relation between soil moisture tension and equivalent pore diameter (for water at 20 °C)

Table 2. Relation between soil moisture tension, pF, and equivalent pore diameter

SOIL MOISTURE TENSION (cm water)	pF	EQUIVALENT PORE-DIAMETER (micron)
1.0	0.00	3,000
3.0	0.48	1,000
5.0	0.70	600
10.0	1.00	300
30.0	1.48	100
60.0	1.78	50
100	2.00	30
300	2.48	10
500	2.70	6
1000	3.00	3

From Fig.2 and Table 2 it can be deduced that in a soil under moisture equilibrium conditions at a height of 60 cm above the groundwater table where a tension of 60 cm water exists, only the pores with an equivalent diameter of 50 micron or less will be filled with water. Lowering the water table from 60 to 100 cm below the point in question will cause a withdrawal of water from the pores with diameters between 50 and 30 micron.

23.1.3 MOISTURE RETENTION CURVES

The graph giving the relation between soil moisture tension and soil moisture content is called moisture retention curve or soil moisture characteristic. If the tension is expressed as the logarithmic value of cm water, the graph is referred to as a pF-curve. Moisture retention curves are used

- to determine an index of the available moisture in soil (the portion of water that can be readily absorbed by plant roots) and to classify soils accordingly, e.g. for irrigation purposes,
- to determine the drainable pore space (effective pore space, effective porosity, specific yield) for drainage design,
- to check changes in the structure of a soil, e.g. caused by tillage, mixing of soil layers, etc.,
- to ascertain the relation between soil moisture tension and other physical properties of a soil (e.g. capillary conductivity, thermal conductivity, clay- and organic matter content).

Figure 3 shows pF-curves of various soil types.

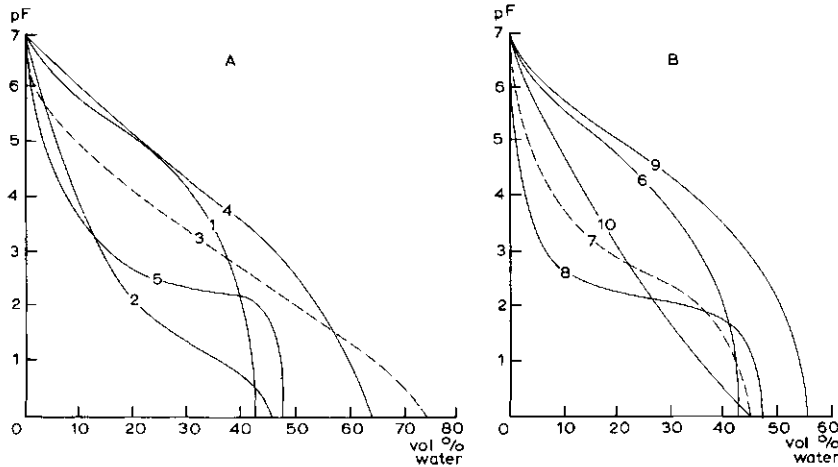


Fig.3. Moisture retention curves of some soils

No.	Country	Region	Soil type	Depth
1	Tunesia	Medjerdah Valley	Heavy clay	40-50
2	Brazil	Amazon Valley	Sandy yellow latosol	100
3	Jamaica	Black River Project	Peat	0-20
4	Senegal	Basse Casamana	Marine (sal.)clay deposit	5
5	Argentina	Albardon	River levee clay	20-40
6	Congo	Lufira	Clay	-
7	E.Pakistan	Ganges alluvium	Sandy loam	20
8	E.Pakistan	Ganges alluvium	Sand	-
9	India	Bombay coastal plain	Salty clay	40-60
10	Brazil	Amazon Valley	Podsol.red fine sandy clay	100

Clay soils show a slow and regular decrease in water content with increasing pF tension (Curves 1, 4, 6 and 9 in Fig.3).

Sandy soils may show only a slight decrease in moisture content in the lower pF range till the point where only a small rise in pF causes a considerable discharge of water due to a relatively large number of pores in a particular diameter range (Curves 5 and 8; range 5-30 and 5-100 micron respectively).

The intersection point of the curves with the volumetric water content axis (tension: 1 cm water, pF = 0) gives the water content of the soils under nearly saturated conditions, which means that this point almost indicates the total pore space percentage (if no air entrapment has taken place). The zero moisture content is based on the oven-dry condition (105 °C), corresponding to a pF of approximately 7.

The total pore space percentage or porosity (ϵ) - defined as the total volume percentage of the total bulk not occupied by solid particles - can also be calculated from the dry bulk density and particle density

bulk density (ρ_d) = mass (grams) per unit volume (1 cm³) of soil including pore spaces

particle density (ρ_s) = mass (grams) per unit volume (cm³) of soil solids.

Now the following equation holds

$$100 \times \rho_d = (100 - \epsilon) \times \rho_s \quad (4)$$

or

$$\epsilon = 100 - \frac{100 \times \rho_d}{\rho_s} \quad (5)$$

23.1.4 COMMENTS ON SOME SOIL MOISTURE CONCEPTS

Field capacity

Field capacity is still often defined as the water content remaining in a soil 2 or 3 days after having been saturated and after drainage by gravity forces has become "negligible". Other definitions have been based on a certain relationship between the water content at field capacity and a particular soil moisture tension, e.g. 1/3 atmosphere.

Older literature mentions the "moisture equivalent" as being an indication for field capacity. The "moisture equivalent" is the weight percentage of water retained by a previously saturated sample of soil 1 cm in thickness after it has been subjected to a centrifugal force of one thousand times gravity for 30 minutes and should correspond with a moisture tension of about 1/3 atmosphere.

All these definitions are misleading. The "practically ceased drainage" concept may only hold for a homogeneous sandy profile having good water-transmitting properties. The attempts to correlate field capacity with a particular moisture tension ignore the fact that in a soil profile the moisture condition is not only dependent on the water-retaining forces but also on the water-transmitting properties over the whole soil profile.

The downward flow (q) of water per unit cross-sectional area is

$$q = K_h \left(\frac{dS}{dz} + 1 \right) \quad (6)$$

where

S_m = soil moisture tension (cm water)

z = depth (cm)

K_h = capillary conductivity (cm/day)

The term $\frac{dS_m}{dz}$ represents the soil moisture tension gradient, the term 1 represents the gravitational gradient and

the term $\frac{dS_m}{dz} + 1$ represents the total hydraulic gradient.

In a well-drained soil the moisture distribution is relatively uniform. Consequently the hydraulic gradient plays a relatively minor role compared with that of the capillary conductivity, which then mainly determines the drainage rate. Any correlation between field capacity and a particular moisture tension will be determined by the capillary conductivity - tension relationship.

For a poorly drained soil the hydraulic gradient is of more importance for the determination of the field capacity tension value. Moisture equilibrium conditions between matric and gravity forces may not be reached even within a long period of time due to the low drainage rate.

Field capacity expressed as moisture percentage on a dry-weight basis is usually considered to be the upper limit of the available water in a soil.

If there is no influence of groundwater, the field capacity is assumed to be roughly approached by a tension of 100 to 200 cm of water (pF 2.0 to 2.3). With a shallow water table the tension at field capacity (cm water) may equal the height (cm) above that table (e.g. pF 1.9 at 80 cm above the water level). Heterogeneity of the soil profile, impeding layers, fluctuating groundwater levels complicate the field capacity problem. Therefore, research on field capacity conditions in the field is preferable to measurements of moisture relationships in the laboratory.

Wilting point or wilting percentage

In text books on plant physiology the (permanent) wilting percentage is defined as the soil moisture condition at which the leaves undergo a permanent reduction of their moisture content (wilting) because of a deficient supply of soil water, a condition from which the leaves do not recover in an approximately saturated atmosphere. Only by the addition of water to the soil will the plant recover.

This characteristic moisture value expressed as moisture percentage on a dry-weight basis indicates the lower limit of the available water in the soil. On the basis of many investigations it can be concluded that the moisture content of an initially wet soil, brought into equilibrium with a pressure of 15 atmospheres (pF 4.2) - e.g. in a pressure plate or pressure membrane apparatus (see Section 2.2) - is a good approximation of the wilting point.

Readily available moisture

This is usually regarded as the amount of water between field capacity and wilting point held by a soil in its rootzone. The readily available moisture is an important parameter for use in irrigation schemes. It is low in coarse sandy soils, higher in clay soils, and maximal in loamy and silty soils (Figs.4 and 5).

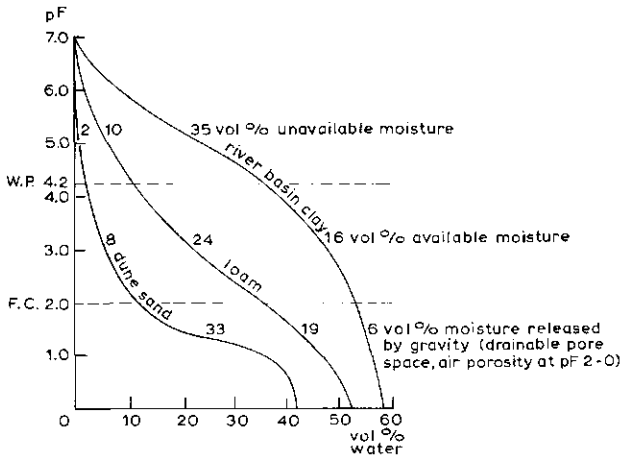


Fig.4. Soil moisture characteristics or pF curves

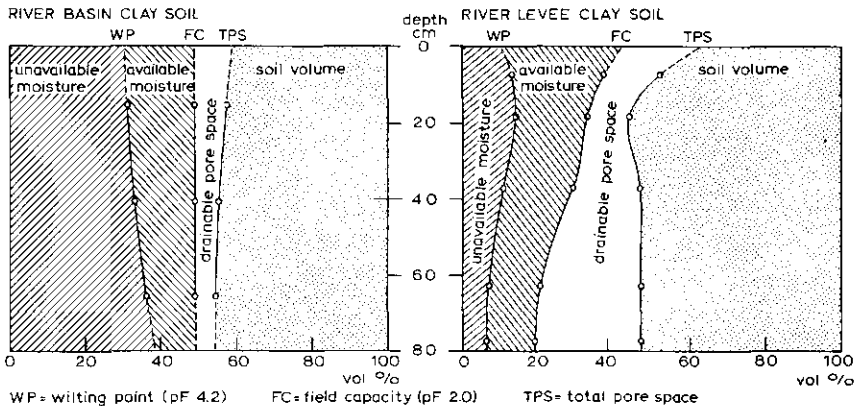


Fig.5. Soil moisture retention of two clay soil profiles

Aeration porosity

This is also called air porosity, non-capillary porosity, aeration capacity, or air capacity. In drainage practices the terms drainable pore space and effective porosity are also used (see Chap.6.5.3, Vol.1). The first mentioned terms are defined as the proportion of bulk volume that is filled with air under a specified moisture tension. The aeration porosity is usually taken to be the aggregate of large pores, drained by a tension of no more than 100 cm of water.

Generally if the aeration porosity amounts to 10 or 15 vol.% or more at $pF = 2.0$, aeration is satisfactory for plant growth. The river basin clay in Figs.4 and 5 is deficient in aeration.

Hysteresis

The relationship between moisture tension and moisture content of a porous material (e.g. soil) is in general not unique. At a given intermediate tension a soil will contain less water if that tension is approached by wetting of a dry soil than if it is approached by drying the same soil starting from a wet state. This phenomenon is called hysteresis (Fig.6B).

The hysteresis effect may be attributable to

- the fact that pores have a larger diameter than their apertures or necks. An empty pore system (A in Fig.6A) will only be completely filled with water to a height inversely proportional to the diameter of its largest pores (Eq.2). The water-filled pore system (B in Fig.6A) will not lose its water until the tension has reached a higher value in accordance with the dimensions of its narrowest apertures,

- variations in packing due to re-arrangement of soil particles by wetting or drying,

- incomplete water uptake by soils that have undergone irreversible shrinking or drying (some clay and peat soils),

- entrapped air.

Due to this hysteresis effect soil moisture-tension relations depend on their past moisture regime. Since moisture retention curves are generally determined in the laboratory as desorption curves (from wet to dry conditions), this being the most efficient procedure, one has to keep in mind that under field conditions the moisture-tension relationship for a given soil is generally not constant (Fig.7).

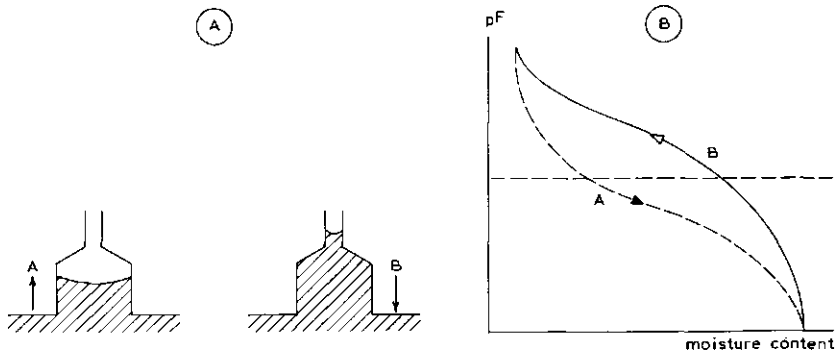


Fig.6. Hysteresis due to pore geometry

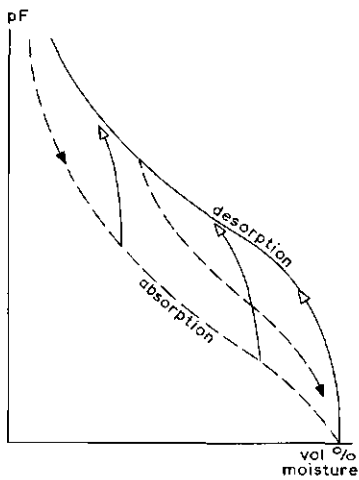


Fig.7. Hysteresis family of pF curves for a certain soil

23.2 OBTAINING SOIL MOISTURE RETENTION CURVES

23.2.1 TAKING A SOIL SAMPLE

Undisturbed soil samples should be taken for the lower tension range, where the structure is of influence on the water-retaining properties. For heavy clay soils this holds for the range $pF = 0$ to approximately $pF = 3.5$; for sandy soils the tension range is smaller. For the higher pF -range disturbed samples are generally used. The water remaining at this tension is found as thin films around the soil particles and filling small pores < 1 micron, which remain intact in spite of disturbance of the macro-structure.

Undisturbed soil samples for pF-measurements are taken in the field at least in duplicate. They can be obtained by pushing stainless steel cylinders (inside diameter 50 mm, height 51 mm, contents 100 cm³) horizontally or vertically into the distinct profile horizons, exposed by digging a pit and subsequently retrieving the filled cylinder. If no pit is dug, an auger in which the same type of cylinder is fixed can be used.

Care should be taken in transporting the samples to the laboratory as they must remain intact. In The Netherlands the sample cylinders are usually transported in wooden boxes lined with foam plastic and rubber plate and each box contains up to 24 cylinders. A plastic bag is filled with loose soil from each profile horizon for the determination of pF-values > 3.

23.2.2 METHODS OF DETERMINATION

To construct the moisture retention curve of a sample, the moisture content of that sample must be measured. This is done by equilibrating the moist soil sample at a succession of known pF values and each time determining the amount of moisture that is retained. If the equilibrium moisture content (expressed preferably as volume percentage) is plotted against the corresponding tension (pF), the moisture retention curve (pF-curve) can be drawn.

There is no single method of inducing the whole range of tensions from pF = - ∞ (total saturation) to pF = 7 (oven-dry) but the following two laboratory methods cover the range satisfactorily.

Porous medium method

The principle of the porous medium method is the equivalence of soil moisture tension to a positive or negative pressure applied to a water-saturated porous medium on which the sample is placed in close contact. The equilibrium moisture content is measured by weighing. This method is the most generally used because it covers the range pF 0 to 4.2, which is of particular importance in agricultural practice. The method is technically simple, is inexpensive, and is well suited for routine analysis.

For the range below 1 atmosphere (pF < 3) it is possible to apply a negative pressure by using a vacuum pump. In case of low tensions (< 200 cm) a water column can be suspended from the water in the porous medium.

It makes no difference in the results whether one uses negative or positive pressure. For the range above 1 atmosphere a positive pressure is applied via

a reducing valve with a compressor unit or with a cylinder filled with compressed air or nitrogen.

The porous medium itself should meet the following requirements:

- it must be possible to apply the required tension or pressure without reaching the air-bubbling pressure (air entry value). This is the pressure at which air bubbles start to leak through the medium, which is then no longer water saturated. The maximum diameter of the pores is the limiting factor for this pressure;

- the water permeability of the medium has to be as high as possible. This demands a homogeneous pore size distribution, matching the applied pressure;

- the medium should not be easily damaged.

Types of porous media

Types of porous media used for the determination of moisture retention curves and the air bubbling pressures of these media are given in Table 3.

Table 3. Maximum tension (cm water or pF) attainable with some water-saturated porous media

MATERIAL	MAXIMUM TENSION (cm water)	MAXIMUM TENSION (pF)
blotting paper	60	1.8
asbestos	150	2.2
Blokzijkl sand	150	2.2 ⁺⁾
kaolin (China clay)	2,500	3.4 ⁺⁾
sintered glass	3,500	3.5
cellulose filters	10,000	4.0
ceramic	15,000	4.2
cellophane		
sausage casing	100,000	5.0

⁺⁾ see Fig.8

Commonly used in laboratories in The Netherlands are the following types of porous media (Fig.8):

- "Blokzijl" sand for the tension range 1 to 100 cm water (pF from 0 to 2.0). (Blokzijl is a small village in the North East Polder of The Netherlands. The main fraction of the extremely fine sand found there is between 35 and 75 micron)

- "Blokzijl" sand with a top layer of kaolin for the range 100 to 500 cm water (pF 2.0 to 2.7)

- Cellophane (Permeable Transparent No.600) for the range 1,000 to 15,000 cm water (pF 3 to 4.2).

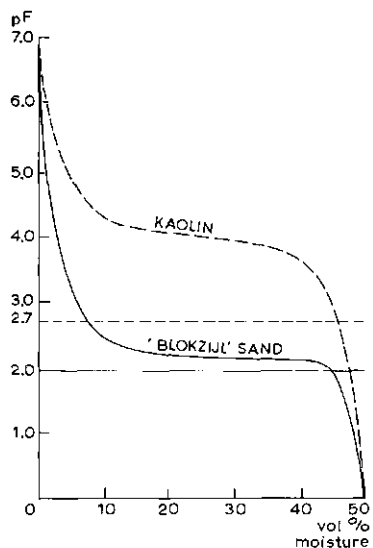


Fig.8. Moisture retention curves of "Blokzijl"-sand and of kaolin

Apparatus for the tension range up to pF 2

The apparatus consists of a box made of stainless steel plate or earthen-ware with perforated P.V.C. conduit pipes for drainage at the bottom. The box is filled with Blokzijl sand. With a levelling bottle connected to the outlet of the drainage pipes, the "water table" can be adjusted within the range of 0 to 100 cm below the surface of the sand (Fig.9).

Generally, tensions of successively 3.2, 10, 31.6, and 100 cm of water column are applied (pF 0.5, 1.0, 1.5 and 2.0). The equilibrium moisture content of the soil samples, to be determined by weighing, is attained within 3-10 days (heavy clay soils may require more time).

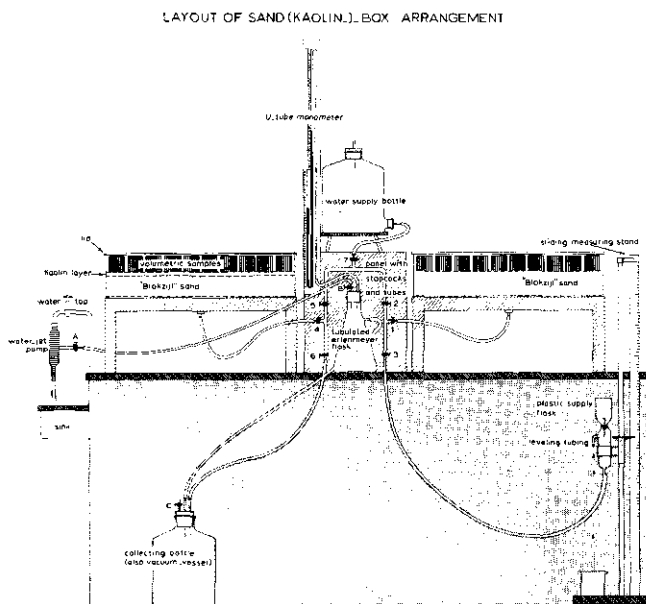


Fig.9. Porous medium application of a tension of $pF = 0$ to 2.7

Apparatus for the tension range of $pF = 2.0$ to 2.7

For the tension range 100 to 500 cm water column ($pF = 2.0$ to 2.7) the apparatus described above is used, the only difference being a supplementary layer of kaolin, 1 to 2 cm thick, applied on top of the sand. Tensions of 200 cm (pF 2.3) and 500 cm (pF 2.7) are realized by means of a water-jet pump or vacuum pump (Fig.9).

Equilibrium moisture content is determined by weighing and is attained in 6 to 16 days, depending on soil type.

Apparatus for the tension range of $pF = 3.0$ to 4.2

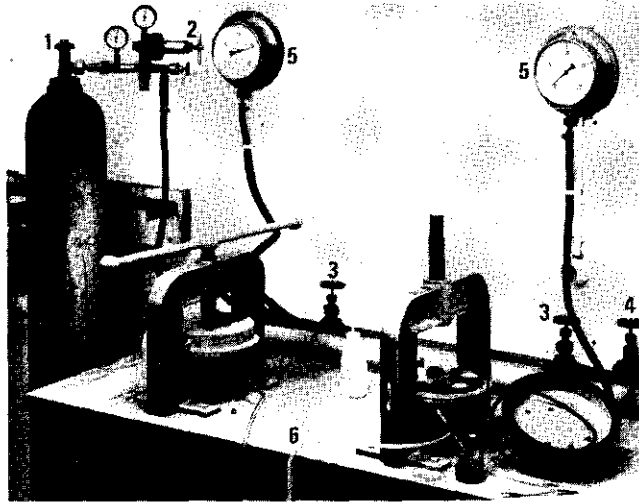
For the tension range 1 to 15 atmospheres a pressure membrane apparatus with cellophane membrane is used (Fig.10). Pre-saturated soil samples are transferred into small rings (height 1 cm, diameter 3 to 4 cm) that have been placed on the cellophane membrane, after which the pressure is applied. Equilibrium is attained in 2 to 12 days, depending on soil type.

An example of tabulation and calculation of moisture contents obtained with the porous medium technique is given in Table 4.

Table 4. Example of tabulation and calculation of the weighing data obtained during the determination of moisture retention curves

EXPERIMENTAL FIELD "SINDERHOEVE", RENKUM, THE NETHERLANDS							
PLOT E _n		SANDY SOIL		CROP: WHEAT			
Date	pF	Gross weight (cylinder included) (gram)					
		Volume of cylinder 100 cm ³					
		5-10 cm depth		Mean	35-40 cm depth		Mean
		Nr. of cylinder			Nr. of cylinder		
		929	985		943	2082	
24.7.'63	INITIAL	253.0	251.3		263.5	261.0	
26.7.'63	0.5	278.3	277.0		287.7	285.4	
29.7.'63	1.0	277.5	276.0		286.3	283.6	
31.7.'63	1.0	277.4	276.0		286.0	282.5	
2.8.'63	1.0	277.3	276.0		285.9	282.4	
6.8.'63	1.5	272.4	272.4		270.4	266.8	
9.8.'63	1.5	272.2	272.1		269.9	266.1	
12.8.'63	1.5	271.9	271.9		269.6	265.8	
14.8.'63	1.5	271.8	271.7		269.6	265.6	
19.8.'63	2.0	263.1	261.3		261.4	256.7	
21.8.'63	2.0	262.9	261.1		261.3	256.6	
26.8.'63	2.0	262.7	260.9		261.2	256.5	
28.8.'63	2.0	262.4	260.5		261.2	256.4	
30.8.'63	2.0	262.4	260.5		261.2	256.4	
3.9.'63	2.3	259.7	257.2		259.0	255.3	
5.9.'63	2.3	258.1	255.4		257.7	253.6	
9.9.'63	2.3	257.8	255.3		257.4	253.2	
11.9.'63	2.3	257.6	255.1		257.2	253.2	
16.9.'63	2.7	256.1	253.7		256.4	252.2	
18.9.'63	2.7	255.4	253.2		255.9	251.8	
20.9.'63	2.7	255.2	252.9		255.7	251.6	
23.9.'63	2.7	255.1	252.7		255.5	251.4	
26.9.'63	2.7	255.1	252.6		255.4	251.4	
30.9.'63	OVEN-DRY WEIGHT (gram)	233.7	231.6		246.2	243.6	
	WEIGHT OF CYLINDER (gram)	98.5	98.9		100.0	91.8	
	NET OVEN-DRY WEIGHT (gram)	135.2	132.7	134.0	146.2	151.8	149.0
	BULK DENSITY	1.35	1.33	1.34	1.46	1.52	1.49
	pF	Volume percentage of water					
	INITIAL	19.3	19.7	19.5	17.3	17.4	17.4
	0.5	44.6	45.4	45.0	41.5	41.8	41.7
	1.0	43.6	44.4	44.0	39.7	38.8	39.3
	1.5	38.1	40.1	39.1	23.4	22.0	22.7
	2.0	28.7	28.9	28.8	15.0	12.8	13.9
	2.3	23.9	23.5	23.7	11.0	9.6	10.3
	2.7	21.4	21.0	21.2	9.2	7.8	8.5
	3.4			13.3 ⁺			3.3 ⁺
	4.2			8.4 ⁺			2.7 ⁺
	6.0			3.1 ⁺			0.9 ⁺

⁺) calculated from weight % of water × bulk density (see Section 3.1)



- | | |
|--|------------------|
| 1 VALVE OF GAS CYLINDER | 4 OUTLET VALVE |
| 2 ADJUSTING SCREW OF REDUCING VALVE | 5 PRESSURE-GAUGE |
| 3 VALVE BETWEEN CONDUIT PIPE AND APPARATUS | 6 BURETTE |

Fig.10. Pressure membrane apparatus

Determining the air-bubbling pressure

The air bubbling pressure is the negative or positive pressure at which a water-saturated porous material starts to transmit air. To examine the tension range for which a porous medium is suited, the air-bubbling pressure must be ascertained. This is done by applying a gradually increasing pressure to the pre-saturated porous medium, which is covered by a thin layer of water (Fig.11).

At the moment air bubbling is discerned the corresponding pressure (h_a) is read on a manometer. As air will first pass through the pore with the greatest throat, the equivalent maximum pore diameter (d) can be calculated from Eq.2 as

$$h_a = \frac{3,000}{d}$$

where h_a is expressed in cm water and d in microns (10^{-4} cm).

If the maximum pore diameter of ceramic, sintered glass, steel, and other filters is stated by the manufacturer, the air bubbling pressure can be calculated directly.

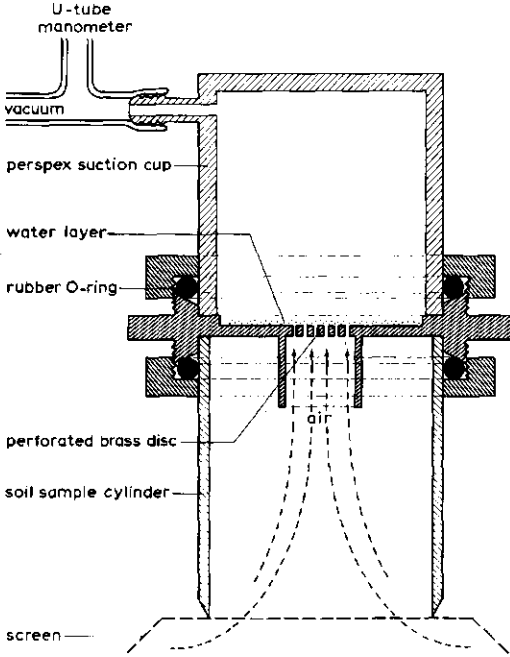


Fig.11. Apparatus to determine air bubbling pressure of soils

Air bubbling pressure of soils

If the air bubbling pressure of homogeneous sandy soils in which the water is mainly capillary-bound is determined, there is a good agreement with the flex-point in the pF-curve where, with increasing tension, the moisture content drops rapidly (Fig.12A). The air bubbling pressure also coincides rather well with the point where the capillary (unsaturated) conductivity begins to decrease rapidly with increasing tension (Fig.12B), (Chap.5, Vol.I).

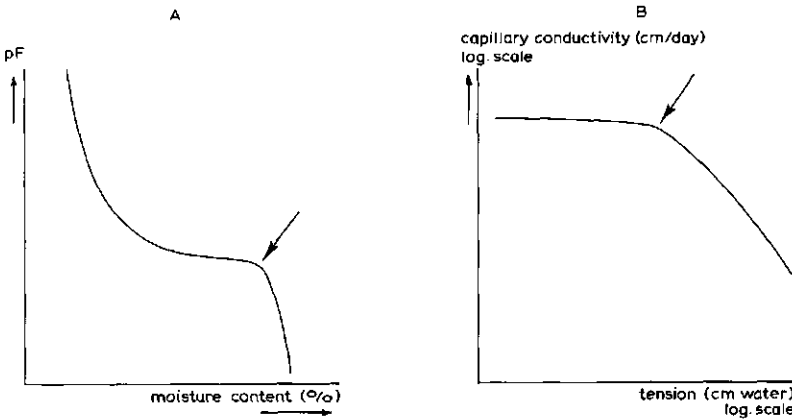


Fig.12. Air bubbling pressure on soil moisture characteristics. A: Air bubbling pressure indicates flex-point of pF-curve. B: Air bubbling pressure indicates transition from saturated to unsaturated conductivity

Vapour pressure method

The principle of the vapour pressure method is based on the relation between soil moisture tension and the water vapour pressure (relative humidity) of the surrounding atmosphere.

With the porous medium method it is the soil moisture tension equilibrium which is measured, since the medium is equally permeable for water or for salt ions and no difference in salt concentration occurs between the released and the retained water. With the vapour pressure method the total moisture tension equilibrium (S_t) is measured as the sum of soil moisture tension (S_m) and solute tension (S_s), because the dissolved soil solutes influence the equilibrium vapour pressure.

The sample under test is allowed to reach moisture equilibrium with a known humidity, transfer of water taking place in the vapour phase. The equilibrium moisture content of the sample is determined by weighing. Since humidity conditions are dependent on temperature, accurate temperature control is needed.

When the soil water comes into equilibrium with the water vapour of the surrounding atmosphere, the following relation exists

$$S_t = S_m + S_s = - \frac{RT}{gM} \ln \frac{e}{e_{sat}} \quad (7)$$

where

- S_t = total tension of soil solution (cm)
- S_m = soil moisture tension of soil solution (cm)
- S_s = solute tension of soil solution (cm)
- R = universal gas constant (8.315×10^7 ergs mole⁻¹ °K⁻¹)
- T = absolute temperature (°Kelvin) ($0^\circ\text{C} = 273^\circ\text{K}$)
- g = acceleration due to gravity (981 cm sec^{-2})
- M = molecular weight of water (18.0 g mole^{-1})
- e = actual vapour pressure
- e_{sat} = saturated vapour pressure at same temperature

Since the relative vapour pressure e/e_{sat} equals $\frac{U}{100}$ (U being the relative humidity of the atmosphere expressed in percentage) Eq.7 becomes

$$S_t = - \frac{RT}{gM} \ln \frac{U}{100} = \frac{RT}{gM} 2.30 \log \frac{100}{U} \quad (8)$$

If S_s can be neglected (non-saline soils), then S_t equals S_m and the following relation exists

$$pF = \log S_m = \log \left(2.3 \frac{RT}{gM} \right) + \log (2 - \log U) \quad (9)$$

which, for 20 °C results in

$$pF = 6.502 + \log (2 - \log U) \quad (10)$$

Equation 10 is illustrated in Fig.13.

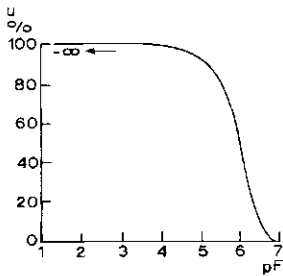


Fig.13. Relation between pF and relative humidity (%) at 20 °C

From Eqs.9 or 10 it can be deduced that the tension range (pF 2 to 4.2) of the available water corresponds with very high relative humidities of 99.99 to 98.85% (see Fig.13). This means that the slightest lowering of temperature may cause condensation of water vapour.

Therefore, in spite of extremely accurate temperature controls and measuring techniques the vapour pressure method, requiring constant atmospheric humidity, can only be applied for the small upper part of the available water range.

Temperature control of approx. 0.01 °C for the range pF 4 to 4.5 and of approx. 0.1 °C for the range pF 4.5 to 5 is required. With the present techniques it may be possible to extend the vapour pressure method down to about pF 3.5.

Salt solutions, sulfuric acid-water or glycerol-water mixtures of known concentration are used to create an atmosphere with constant humidity. The equilibrium vapour pressure above these solutions depends on their concentration.

The volume of the solution or mixture should be large in comparison with the volume of the soil sample so as to restrict its change in concentration due to water uptake by or release from the sample. Therefore it is preferable to use saturated salt solutions (which show no change in concentration as long as both liquid and solid phase are present), especially those whose solubility changes only slightly with temperature. Some examples are given in Table 5.

Table 5. Relative humidity and pF of some saturated salt solutions at 20 °C

SALT	EQUILIBRIUM RELATIVE HUMIDITY (%)	pF
ammonium oxalate	98.8	4.2
potassium sulfate	97.1	4.6
potassium chromate	88.0	5.2
sodium chloride	75.8	5.6
potassium carbonate	44.0	6.0

Thermocouple technique

New techniques have opened up possibilities for more accurate measurements of high relative humidities of approx. 95 to 99.98%, corresponding with a pF-range of 4.8 to 2.5. For such measurements a soil sample is placed in a small humidity chamber in which the vapour pressure of the atmosphere comes into equilibrium with the vapour pressure of the soil water.

A promising measuring technique is to send a small electric current through a thermocouple, fixed above the sample, in the direction that will cause cooling to set in (Peltier-effect) to below the temperature at which the water vapour will condense (dew-point). A thin water film will be formed at the thermocouple junction, which acts temporarily as a wet-bulb thermometer. When the current is broken off, the water film evaporates, causing a temperature difference between the wet and the dry bulb - the latter consisting of the incoming wires of the thermocouple - which depends on the vapour pressure (humidity) of the surrounding atmosphere. The temperature difference in the thermocouple generates a proportional minute thermal electromotive force that can be amplified and measured with a microvoltmeter to which a line recorder can be coupled (Fig.14).

Calibration of the output can be made above molar sodium- or potassium-chloride solutions. If from a series of various moisture contents of the same soil sample the corresponding vapour pressures are measured, it is possible to assess a part of the moisture retention curve. More research will still be needed to overcome all technical problems, which up to now have impeded a routine application of this method.

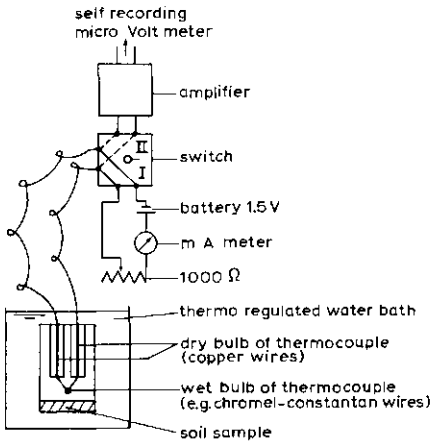


Fig.14. Thermocouple measuring circuit

23.3 MEASURING SOIL MOISTURE CONDITIONS

Several methods of measuring the soil moisture content or tension, either in the field or in the laboratory, are known. Only the most frequently applied methods will be discussed.

23.3.1 GRAVIMETRIC DETERMINATION OF SOIL MOISTURE CONTENT

Measuring the water content by the gravimetric method involves weighing the natural (wet) soil sample, and reweighing it after the water has been removed by drying to constant weight in an oven at 105 °C. The difference in weight indicates the moisture content and is expressed either as a percentage of the weight or volume of the oven-dry soil. The weight-percentage of moisture can be calculated as follows

$$\begin{aligned} \text{weight \% of moisture} &= \frac{\text{weight wet soil} - \text{weight oven-dry soil}}{\text{weight oven-dry soil}} \times 100 \\ &= \frac{\text{grams water}}{\text{grams oven-dry soil}} \times 100 \end{aligned}$$

For irrigation, drainage, and other agronomic purposes it is useful to know the moisture content on a volume basis (vol.% of water = mm of water per 10 cm depth).

If soil samples of known volume are taken, the moisture content expressed as a percentage of that volume can be calculated as follows

$$\begin{aligned} \text{vol \% of moisture} &= \frac{\text{weight wet soil} - \text{weight oven-dry soil}}{\text{volume soil}} \times 100 \\ &= \frac{\text{cm}^3 \text{ water}}{\text{cm}^3 \text{ soil}} \times 100 \end{aligned}$$

Another way to obtain the soil moisture content on a volume basis is to convert the weight percentage into volume percentage by multiplying the first with the dry bulk density (volume weight) of the soil

$$\text{volume \% of moisture} = \text{weight \% of moisture} \times \text{dry bulk density}$$

with the dry bulk density defined as the mass of dry soil per unit bulk volume

$$\text{bulk density} = \frac{\text{mass oven-dry soil}}{\text{volume soil}} = \frac{\text{grams oven-dry soil}}{\text{cm}^3 \text{ soil}}$$

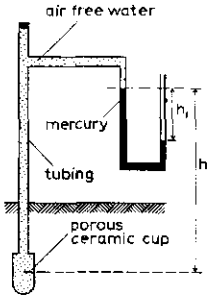
Although the gravimetric determination of soil moisture content is rather laborious, it is, because of its simplicity and reliability, the most extensively applied technique and is used as calibration standard for other methods.

23.3.2 TENSIO METER

A tensiometer consists of a porous cup positioned in the soil and attached to a tube which is connected to a vacuum gauge, (Bourdon manometer or a U-tube mercury manometer, Fig.15). Cup and tube are filled with water. Water flows into or out of the cup through the cup wall as long as there is a moisture tension gradient between the water in the cup and that in the soil. Under equilibrium conditions the manometer indicates the ambient soil moisture tension. Practical use is restricted to the range 0 to 800 cm water tension ($pF < 2.9$) because with higher tensions air leaks through the wall of the porous cup. To what extent the available water range, expressed as a percentage of the available water between pF 2.0 and 4.2, is covered by the tensiometer depends on the shape of the pF -curve as is shown for three soil types in Fig.16.

The best way to position the tensiometer in the soil is to bore a hole to the required depth with a small auger of approximately the same diameter as the porous cup and to push the cup into the bottom of the hole.

The soil used to refill the hole around the tubing has to be well compacted. Due to thermal gradients between the soil and the above-ground parts of the instrument the measurements may show variations. Therefore it is advisable to shield the tensiometer against radiation of the sun, to use plastic instead of metal tubing, and to read the manometer regularly at the same hour of the day, e.g. in the early morning.



equilibrium tension =
 $= (13.6 h_1 - h)$ cm water

Fig.15. Tensiometer

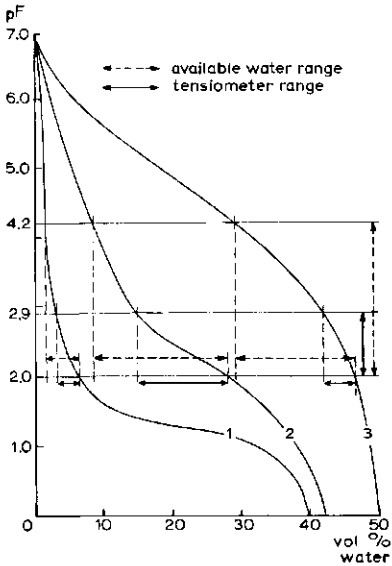


Fig.16. Part of the available moisture range covered by tensiometer, depending on soil type

1. Sand 50% of available moisture
2. Loam 75% of available moisture
3. Clay 25% of available moisture

23.3.3 ELECTRICAL RESISTANCE UNITS

The principle of the electrical resistance unit is based on the change in electrical resistance in a porous material due to a change in moisture content. Resistance units consist of two parallel electrodes - embedded in gypsum, nylon, fibreglass, or a combination of gypsum with nylon or fibreglass - which are placed in the soil. The resistance to an electrical current is dependent on the moisture condition of the unit, which itself is in moisture tension equilibrium with the surrounding soil. It can be measured by means of a Wheatstone bridge.

The electrical resistance of the unit should be calibrated against the soil moisture tension (Fig.17). Calibration can be done in the field by taking soil samples for moisture content determination at the same time as resistance measurements are made. If the moisture retention curve is known, the relation between resistance and tension can be plotted. For calibration in the laboratory either the same procedure can be followed or the resistance-tension relation can be measured on soil samples subjected to tensions by means of the porous medium method. If the blocks are of standard manufacture, it is not necessary to calibrate each unit separately.

Resistance readings will be affected by hysteresis phenomena and by the presence of electrolytes in the soil solution. The lowering of the resistance, due to electrolytes, is counteracted by the saturated solution of calcium sulphate present in gypsum blocks. The blocks can therefore be used in slightly saline soils of which the saturation-extract shows a conductivity of less than approximately 2 mmhos cm^{-1} . Under wet conditions and in acid soils the gypsum block will gradually deteriorate. Nylon and fibreglass units are more durable but are very sensitive to electrolytes.

The electrical resistance method is not suitable for soils showing shrinkage (contact problems) or for sandy soils with a somewhat horizontal section in the moisture retention curve for which a slight change in tension would cause a great change in water content.

Gypsum blocks are recommended for the range of pF 2.3 to 4 (Fig.17), nylon elements encased in gypsum for the range of pF 1.3 to 2.3.

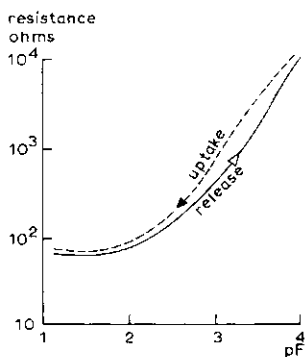


Fig.17. Electrical resistance (ohms) of gypsum block versus moisture tension

23.3.4 NEUTRON SCATTERING

The neutron-scattering method is based on the fact that fast-moving neutrons emitted by a radio-active source are slowed down by collisions with the nuclei of the soil and can be counted by a detector. Because the moderating ability of the soil nuclei (with the exception of boron and chloride) is small compared with that of hydrogen, which has the same mass and size as the neutron, and since the hydrogen is mainly present in water, the measured density of slow neutrons is an indication of the (volumetric) moisture content of the soil.

The portable equipment consists of a probe unit and a scaler. The probe, containing a neutron source either at its end or at its side, is inserted into an access tube in the soil down to the desired depth. The emitted fast neutrons collide with the nuclei in the soil and lose energy. A proportion of the scattered slow neutrons (thermal neutrons) are absorbed in a boron trifluoride gas-filled tube (counter). Ionization of the gas results in discharge pulses, which are amplified and measured by a scaler (Fig.18).

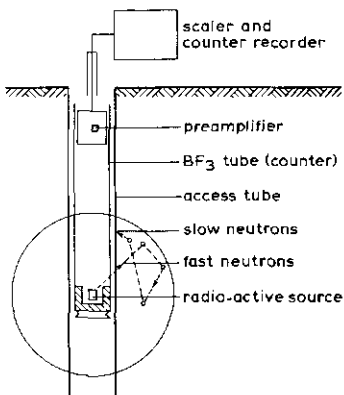


Fig.18. Neutron moisture meter

For a direct comparison of results, size, shape, and material of the access tubes must be identical for each measurement. Aluminium is a frequently employed material as it offers practically no resistance to neutrons; brass, steel, and plastic show a lower neutron transmission. Calibration of the relation between count rate and moisture content (which can not be transferred from one type of apparatus to another) can be done

- in the field by taking volumetric soil samples around the access tube;
- in the laboratory - same procedure with soil in a container;
- by using standards with a range of known constant moisture contents, e.g. sand-alum or sand-paraffin mixtures;
- by comparison with a calibrated meter.

Possible counting drifts with time can be checked in the field or in the laboratory by using water in a container as reference. For depth-samples the radius of the measured soil sphere varies from 15 cm in a wet soil to 30 cm in a dry soil (Fig.19). Special surface probes can be used to measure the water content in the upper surface layer.

The most common neutron source hitherto used is Radium-Beryllium. Recent research, however, has shown that an Americium-Beryllium or a Plutonium-Beryllium source is preferable since the gamma ray emission of these sources, which is hazardous to health, is of much lower intensity. Moreover, greater neutron fluxes can then safely be used, enhancing the accuracy of the measurement.

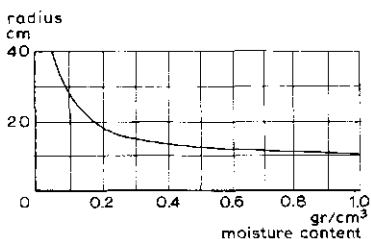


Fig.19. Relation between radius of the measured sphere and the soil moisture content for a neutron moisture meter

23.3.5 GAMMA-RAY ATTENUATION

A radioactive source emitting gamma-rays is inserted in the soil. The gamma-ray technique is based on the fact that the absorption coefficient of elements with an atomic number between 2 and 30 is the same when the same radiation energy is used. Since these elements are predominant in soil, a change in absorption will be due to a change in the wet bulk density of the soil. Assuming that the dry bulk density does not change over long periods changes in adsorption give a direct measure of changes in moisture content.

Generally Cesium¹³⁷ is used as emitter of gamma-rays. If the apparatus is suitably constructed, the gamma-rays can be collimated in such a way that moisture changes in very thin layers (1 cm) can be measured when the transmission or double probe technique is used. With this double-probe technique source and detector are fixed to a frame at a horizontal distance of 40 cm. Source and detector are lowered into the soil through two parallel access tubes (Fig.20). The access tubes are of the same type as those used for neutron-scattering.

Another technique is the scattering technique, where a single probe contains both source and detector, separated by shielding. With this method, for which one access tube suffices, an approximately spherical sample with a diameter of 20 to 75 cm, depending on apparatus and soil characteristics, is measured. Therefore the double-probe technique is better suited for the study of shallow layers in a soil profile.

A comparison of the merits and limitations of methods mentioned in Sect.3.1 to 3.5 is given in Table 6.

The above enumeration of methods for the determination of soil water is not comprehensive. The electrical capacitance, the thermal conductivity, and the porous absorber method have not found wide usage and cannot be recommended at this time. The same can be said for the freezing-point-depression and the centrifuge method in the laboratory.

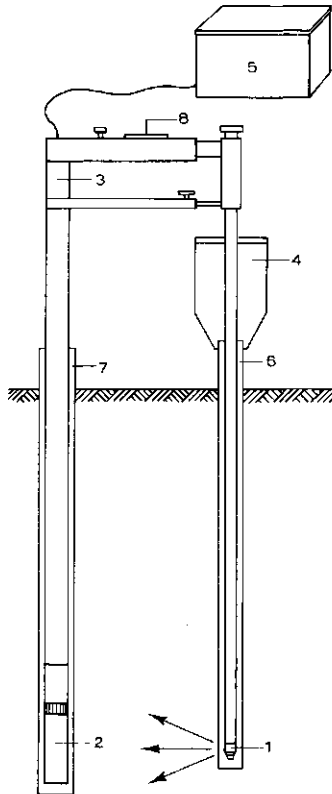


Fig.20. Set up of gamma ray transmission equipment. 1=source, 2=detector, 3= connection frame of detector and source, 4=container, 5=counter, 6=access tube for source, 7=access tube for detector

Table 6. Principal merits and limitations of the common methods of soil moisture measurement

METHOD	MERITS	LIMITATIONS
sampling (gravimetric)	complete moisture range, not sensitive to salt and temperature, reliable (used as calibration standard)	not reproducible, high sampling error, volume weight to be known, laborious
tensiometer	direct measurement of moisture tension, reproducible	limited tension range, hysteresis effect, contact with soil (particularly in swelling and shrinking soils), apparatus fragile
electrical resistance blocks (gypsum) and nylon units)	practically complete moisture range, reproducible	sensitive to temperature and salt (especially nylon units), hysteresis effect, contact with soil, calibration, limited life in wet, acid soils (particularly gypsum units)
neutron scattering	complete moisture range, reproducible, no salt and temperature influence, relatively large horizontal distance	apparatus fragile and expensive, relatively large vertical distance, deviations due to high amounts of organic matter and uncommon excesses of B, Cl, and Fe
gamma radiation adsorption	complete moisture range, reproducible, relatively large horizontal and small vertical distance, no salt and temperature influence	apparatus fragile and expensive, volume weight to be known

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SURVEYS AND INVESTIGATIONS

24. DETERMINING HYDRAULIC CONDUCTIVITY OF SOILS

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PURPOSE AND SCOPE

A number of field and laboratory methods for the determination of hydraulic conductivity are described.

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24.1 INTRODUCTION

24.1.1 DEFINITIONS

The term hydraulic conductivity is the proportionality factor K in Darcy's law for the flow of water in soil (Chap.6, Vol.I),

$$v = - Ki \tag{1}$$

where

v = flow velocity (LT^{-1})

K = hydraulic conductivity (LT^{-1})

$i = \frac{dh}{dx}$ = hydraulic potential gradient (dimensionless)

It can be seen that for a unit hydraulic gradient the flow velocity equals K. The factor K usually stands for the hydraulic conductivity of a saturated soil, i.e. under positive pressure of the soil water. It is considered independent of the pressure gradient because we assume that the water-conducting porosity does not change with the pressure gradient. Under unsaturated conditions, however, the hydraulic conductivity varies with the soil moisture suction, since suction, soil moisture content, and water-conducting porosity are closely interrelated. K is then called capillary conductivity and is represented by the symbol K_h to signify the relationship.

The hydraulic conductivity of a soil represents its average water transmitting properties, which depend mainly on the number and the diameter of the pores present. If these are uniformly distributed, the soil is said to be homogeneous. In such a soil the hydraulic conductivity is the same in all directions and the soil is said to be isotropic.

Soils commonly show a certain stratification so that the hydraulic conductivity in one direction is greater than in another. A soil in which the hydraulic conductivity at any point has preferential directions is called anisotropic. If the anisotropy varies from point to point in a given layer, the layer is said to be heterogeneous. If, on the other hand, the condition of anisotropy is the same from point to point in the layer, the layer is still homogeneous.

Apart from porosity, hydraulic conductivity also depends on the viscosity and density of the soil water. This relationship is expressed in the following equation (see also Chap.6, Vol.I)

$$K = K' \frac{\rho g}{\eta} \tag{2}$$

where

- K = hydraulic conductivity (cm/sec)
- K' = intrinsic permeability independent of density and viscosity (cm²)
- ρ = mass density of the solution (g/cm³)
- g = acceleration due to gravity (cm/sec²)
- η = dynamic viscosity of the solution (poise, g/cm sec)

Density and viscosity are determined mainly by the temperature and salt concentration of the soil water. Only relatively small variations in these factors are found in the field, but corrections may be necessary when laboratory data are used.

24.1.2 OUTLINE OF THE VARIOUS METHODS

Hydraulic conductivity, K, can be determined either from soil samples in the laboratory or from soil bodies in situ. Both methods impose certain flow conditions on a soil body, after which discharge is measured and K calculated with a formula describing the relation between K, the flow conditions, and the discharge. An analytical derivation of the formula is possible because flow conditions are relatively simple to produce in the laboratory. Flow systems with known boundaries are obviously more difficult to create in the field. This has certain implications for the determination and derivation of the geometry factor (C) which is introduced into the formulas. The field measurements are of two types, viz. measurements in non-saturated soils (above the water table) and in saturated soils (below the water table). Examples of the first type are the double tube method, the infiltrometer method, and the inversed auger hole method, which are discussed in Sects. 4.4, 4.5, and 4.6 respectively.

Methods for measuring K below the water table can be roughly classified into large-scale field measurements and point measurements. The former are discussed in Chaps. 21, 25, 26, Vol. III. Point measurements below the water table require a bore hole from which water is removed and the rate of rise of the water level is observed (augerhole method, Sect. 4.1; piezometer method, Sect. 4.2), or a bore hole in which the water table is kept at a certain level by pumping and the water extraction measured (pumped borehole, Sect. 4.3).

Estimation methods, employing the correlation between hydraulic conductivity and the pore or grain size distribution of the soil are discussed in Sects. 2.1 and 2.2 respectively.

24.2 CORRELATIVE METHODS

24.2.1 THE PORE SIZE DISTRIBUTION OR pF-CURVE METHOD

Principle

This method is based on the laws of Poiseuille and Darcy and describes the relation between K and the hydraulic gradient for a given cylindrical capillary pore. An average K value can be estimated with the aid of a moisture retention curve.

Calculations

According to the laws of Poiseuille and Darcy the hydraulic conductivity of a capillary pore for laminar flow is (Chap.6, Vol.I)

$$K = \frac{r^2 \rho g}{8\eta} \quad (3)$$

where

- K = hydraulic conductivity (cm/sec)
- r = radius of cylindrical pore (cm)
- ρ , g, and η are as previously defined

The height to which water will rise in a vertical capillary cylinder is (Chap.5, Vol.I)

$$h = \frac{2\sigma}{\rho g r} \quad (4)$$

where

- h = height of capillary rise or capillary pressure (cm)
- σ = surface tension of water against air (dyne/cm, g/sec²)
- ρ and g are as previously defined.

Combining Eqs.3 and 4 gives

$$K = \frac{\sigma^2}{2\rho g \eta h^2} \quad (5)$$

For water at 20 °C, $\sigma = 73$ dyne/cm, $\eta = 0.01$ poise and $\rho g = 980$ g/cm² sec², whence

$$K = \frac{270}{h^2} \quad (6)$$

MARSHALL (1957) determined the pore size distribution using the soil moisture retention curve (Fig.1). The vertical axis, with the moisture content in vol.%,

is divided in n equal intervals. Each interval i represents a number of capillaries having an average radius equal to the capillary height, h , required to empty the pores.

Marshall subsequently estimated K from

$$K = 270 \frac{\epsilon^2}{n^2} \sum_{i=1}^n \frac{2i-1}{h_i^2} \quad (7)$$

where

- K = hydraulic conductivity (cm/sec)
- ϵ = the porosity of the soil (vol.%)
- n = number of intervals on the moisture content axis
- h = height of capillary rise (cm)
- $2i-1$ = weight factor

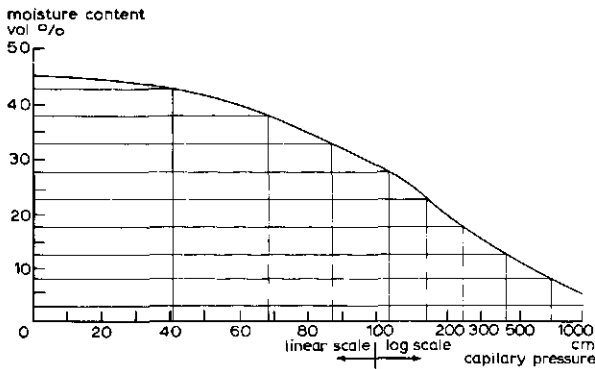


Fig.1. Soil moisture retention curve of a sandy soil with average capillary pressure by porosity class and moisture range

Example 1

The soil moisture retention curve of a sandy soil has been divided along the moisture axis in $n = 9$ equal intervals of 5%. The total porosity of the soil is assumed to equal the soil moisture content at zero capillary pressure, $\epsilon = 0.45$. The average capillary pressure required to lower the soil moisture content to the average moisture content of each interval is given in Fig.1 from which Table 1 is derived.

The hydraulic conductivity is calculated, according to Eq.7

$$K = \frac{270 (0.45)^2}{9^2} \left[\frac{1}{41^2} + \frac{3}{67^2} + \frac{5}{85^2} + \frac{7}{105^2} + \frac{9}{140^2} + \frac{11}{250^2} + \frac{13}{400^2} + \frac{15}{730^2} + 0 \right]$$

$$K = 2.2 \times 10^{-3} \text{ cm/sec or } K = 1.9 \text{ m/day}$$

Table 1. Relation between moisture content and capillary pressure

Porosity class	Moisture range (%)	Average capillary pressure (cm)
1	40 - 45	41
2	35 - 40	67
3	30 - 35	85
4	25 - 30	105
5	20 - 25	140
6	15 - 20	250
7	10 - 15	400
8	5 - 10	730
9	0 - 5	> 1000

Discussion

The pF-curve method for determining hydraulic conductivity is only applicable to soils with a single grain structure such as sands. The continuity and regularity of small pores is probably higher than those of larger pores and to account for this a weight factor $(2i-1)/n^2$ is introduced in Eq.7 (see for the derivation of Eq.7 MARSHALL, 1957). It should be noted that when hydraulic conductivity is calculated from the moisture retention curve only an order of magnitude is obtained and not an exact value.

24.2.2 THE GRAIN SIZE DISTRIBUTION OR SPECIFIC SURFACE METHOD

Principle

The principle of this method is similar to that of the pore size distribution technique, the main difference being that the pore size is not related to capillary pressure but to the specific surface of the soil particles. The specific surface (or U figure) is a single parameter by which the grain size distribution of a sample of sandy soil is expressed.

Calculations

The specific surface (U) is defined as the ratio of the total surface area of the soil particles to the surface area of an equal quantity, by weight, of spherical particles of the same material with a diameter of 1 cm.

If we assume w gram of a material whose particles are spheres with a diameter of d cm and a mass density ρ_s , then the number of particles in w gram material is

$$\frac{w}{1/6 \pi d^3 \rho_s} = \frac{6w}{\pi d^3 \rho_s}$$

and the total surface area of these particles

$$\frac{6w}{\pi d^3 \rho_s} \pi d^2 = \frac{6w}{d \rho_s} \text{ cm}^2$$

The total surface area of w gram of these particles with a diameter of 1 cm is therefore $6w/\rho_s$ cm². The specific surface of the material

$$U = \frac{6w}{d \rho_s} / \frac{6w}{\rho_s} = \frac{1}{d} \quad (8)$$

a relation derived by HOOGHOUDT (1934).

In fact granular soils generally do not consist of uniform particles of one single diameter, but will contain grains of different sizes, to be grouped in fractions, each with certain limits of particle size. The relation between fraction size and U figure is given in Table 2.

Table 2. Sand fractions, their particle size limits, and U figure

Description	Particle size limits (micron)		$U = \frac{1}{d} \left(\frac{1}{d_1} + \frac{1}{d_2} \right)$ d in cm
	d_1	d_2	
Extremely fine sand	16	63	$\frac{1}{2} (625 + 160) = 390$
Very fine sand	63	83	$\frac{1}{2} (160 + 120) = 140$
Fine sand	83	125	$\frac{1}{2} (120 + 80) = 100$
Moderately fine sand	125	200	$\frac{1}{2} (80 + 50) = 65$
Moderately coarse sand	200	333	$\frac{1}{2} (50 + 30) = 40$
Coarse sand	333	500	$\frac{1}{2} (30 + 20) = 25$
Very coarse sand	500	1000	$\frac{1}{2} (20 + 10) = 15$
Extremely coarse sand	1000	2000	$\frac{1}{2} (10 + 5) = 7.5$

For a sand sample, consisting of a number of different fractions, one can calculate the value of U for the total sample as follows

$$U_{\text{tot}} = \frac{\sum U_i W_i}{W_{\text{tot}}} \quad (9)$$

where

W_i = weight of the fraction i

W_{tot} = weight of all the fractions

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The relation between hydraulic conductivity and grain size distribution of sandy material has been the subject of many investigations. If the U figure is chosen as parameter for the grain size distribution, it appears that in all the formulas the hydraulic conductivity K is inversely proportional to $1/U^2$. For homogeneous sands, free from clay, the following equation, which expresses this relationship, is often found in literature

$$K = \frac{C}{U^2} \frac{\epsilon^3}{(1 - \epsilon)^2} \quad (10)$$

where ϵ = porosity of the sand, and C is a factor representing the influence of the shape of the particles and the voids. Various investigations have shown that the proportionality factor between K (in m/day) and $1/U^2$, or in other words the product KU^2 , varied from 31×10^3 to 71×10^3 at a porosity $\epsilon = 0.40$ (see Table 3).

Table 3. Values of KU^2 at an assumed porosity of 0.4

Investigator	$K U^2$ (K in m/day)
Seelheim	31×10^3
Slichter	33×10^3
Krüger	36×10^3
Terzaghi	$40 \times 10^3 - 71 \times 10^3$
Schönwalder	40×10^3
Zunker	$32 \times 10^3 - 44 \times 10^3$
Hooghoudt	44×10^3
Fahmy	47×10^3

ERNST (unpublished research 1955), after comparing the results of laboratory tests with those of field pumping tests, established the following empirical relation

$$K = 54000 U^{-2} C_{so} C_{cl} C_{gr} \quad (11)$$

where

K = hydraulic conductivity (m/day)

U = specific surface of the main sand fraction

C_{so} = correction factor for the sorting of the sand (Fig.2A)

C_{cl} = correction factor for the presence of particles < 16 microns (Fig.2B)

C_{gr} = correction factor for the presence of gravel (Fig.2C)

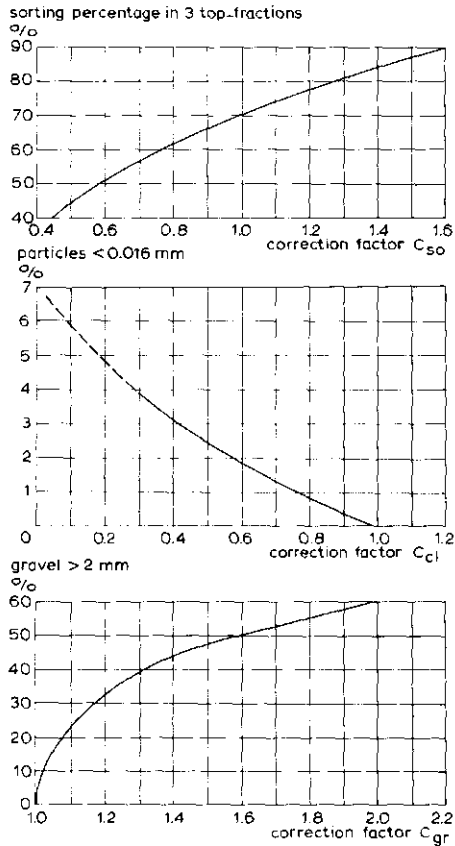


Fig.2. Correction factors for estimating hydraulic conductivity of sands from grain size distribution (U-figure). (Data from ERNST, 1955, unpublished)

The correction factor C_{so} takes into account the sorting of the sand, taken as the percentage by weight of the three best represented, neighbouring subfractions, i.e. the top fractions. For a sorting of 70%, the factor C_{so} equals 1, for a higher sorting $C_{so} > 1$, for a lower sorting $C_{so} < 1$.

The correction factor C_{cl} takes into account the content of particles < 16 microns (percentage by weight of dry soil). It is not possible to use this method to determine K if the soil has more than 6% particles < 16 microns. The method is unreliable for soils having more than 4% particles < 16 microns. The factor C_{cl} is ≤ 1 .

The correction factor C_{gr} takes into account the gravel-content, i.e. particles > 2 mm. When gravel occurs intermixed with the finer particles, it obstructs the

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flow of water and consequently decreases hydraulic conductivity. Normally, however, gravel occurs as separate layers even though it seems intermixed in disturbed samples. The occurrence of gravel signifies that layers of high hydraulic conductivity are present. In the graph for the C_{gr} -factor only this effect has been taken into account. The factor $C_{gr} > 1$.

Clay content, gravel content, sorting and U figure can all be determined from a grain size analysis of a sandy sample, either disturbed (from a bailer) or undisturbed (core sample).

Example 2

The analysis of samples obtained from an aquifer 5 cm thick identifies the sandy material as a fine sand, having a sorting of 60% of the material in the top three fractions, approximately 2% particles < 16 microns, and no gravel. The K value can be computed from Eq.11

$$K = 54000 U^{-2} C_{so} C_{cl} C_{gr} = 54000 \times 100^{-2} \times 0.76 \times 0.58 \times 1.0$$

or

$$K = 2.4 \text{ m/day}$$

Discussion

This grain size method for determining K should be regarded as no more than a refined estimation procedure which may give useful results, when other techniques are impracticable. It is generally used to estimate the K value of deeper-lying and thick aquifers and gives an order of magnitude, not an accurate result. It should preferably be used in conjunction with a pumping test.

The estimated K value when multiplied by D, the thickness of the layer from which the sample is taken, gives the transmissivity of that layer. This procedure is repeated for all consecutive layers sampled and the sum of all the transmissivity values gives the transmissivity (KD) of the aquifer. This result is then compared with that obtained from the pumping test (DE RIDDER and WIT, 1965).

24.3 LABORATORY METHODS

Principle

Laboratory measurements of hydraulic conductivity are conducted on soil samples, contained in cylinders of known dimensions. If the hydraulic conductivity values are to be representative of a soil in situ, undisturbed samples must be obtained. Stainless steel cylinders with a thin wall and one sharpened end are used to extract soil samples above the groundwater table (Kopecky rings usually 100 cm³, 50 mm diameter × 51 mm length, wall thickness 1.5 mm). They are pressed gradually and evenly into the face of a profile pit. Care should be exercised to minimize soil compaction. The soil around the cylinder is then removed and the cylinder containing the sample is withdrawn. The ends of the sample should not be cut with a knife but should be removed to expose the natural structure of the soil. If the profile pit is cut in steps, vertical samples can also be taken.

Undisturbed samples from the zone under the groundwater table can be obtained by using a coring apparatus in a borehole (Fig.3). A tube is fitted into the coring apparatus and driven into the soil at the bottom of the borehole. Closing off the tube above and below the sample with inflatable rubber rings prevents the loss of material during extraction. These samples can only be taken in a vertical direction.

The apparatus required for the measurement of hydraulic conductivity is shown in Fig.4. A constant water level can be maintained in the container and measurements can be made under a constant or a falling head. For further details reference is made to WIT (1967).

24.3.1 CONSTANT HEAD METHOD

Calculations

The principle of the constant head method for a Kopecky cylinder is illustrated in Fig.5. A constant head can be created by means of a siphon and an overflow. The induced steady flow can be measured in a burette.

According to Darcy's law, K can be expressed as (see also Chap.6, Vol.I)

$$K = \frac{QL}{A\Delta h} \quad (12)$$

where

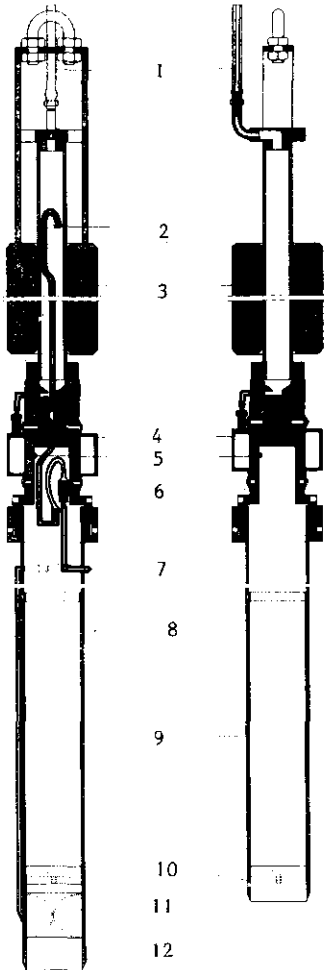


Fig.3. Profile diagram of sampler with shoe A (left) and shoe B (right); the top of the apparatus turned 90 degrees and the tubes in the sampler head omitted: (1) = air hose; (2) = nylon cloth; (3) = steel weight; (4) = rubber tire; (5) = port; (6) = sampler head; (7) = air line; (8) = outer barrel; (9) = sampling tube; (10) = shoe B; (11) = rubber ring; (12) = shoe A. (After WIT, 1962)

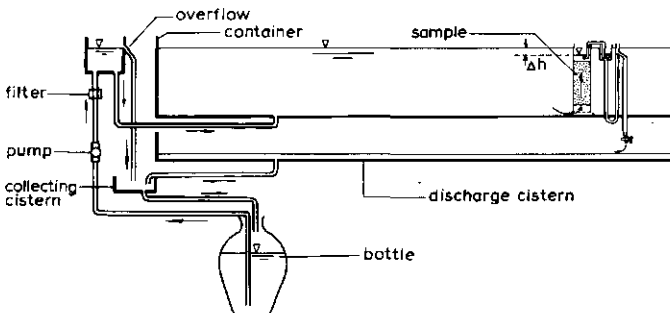


Fig.4. Schematic diagram of apparatus for measuring hydraulic conductivity of undisturbed samples (after WIT, 1967)

- K = hydraulic conductivity (cm/day)
- Q = steady flow (cm³/day)
- Δh = constant head loss (cm)
- L = length of the sample (cm)
- A = cross-sectional area of the sample (cm²)

Example 3

The sample in the Kopecky ring is 5.1 cm long and has a cross-sectional area of 19.6 cm². The constant head loss maintained is 1.0 cm and the constant discharge rate is 190 cm³/day. According to Eq.12 we have

$$K = \frac{190 \times 5.1}{1.0 \times 19.6} = 49 \text{ cm/day} = 0.49 \text{ m/day}$$

24.3.2 FALLING HEAD METHOD

Calculations

The principle of the falling head method is illustrated in Fig.6. A sample taken in a tube 6.2 cm in diameter and 30 cm long is placed in the container shown in Fig.4. After the water level above the sample reaches equilibrium and the soil is saturated, the water on top of the sample is removed and the consequent difference in head causes an upward flow through the sample. As a result the initial head loss and the flow are reduced. Measurements of the rate at which the head loss is reduced are used to calculate the hydraulic conductivity

$$K = \frac{A_1 L}{A_2 (t_2 - t_1)} \ln \frac{h(t_1)}{h(t_2)} \quad (13)$$

where

- K = hydraulic conductivity (cm/day)
- L = length of the sample (cm)
- h(t₁), h(t₂) = head at time t₁ and t₂ respectively (cm)
- t₂-t₁ = time interval (days)
- A₁, A₂ = cross-sectional area of observation tube and soil sample respectively (cm²)

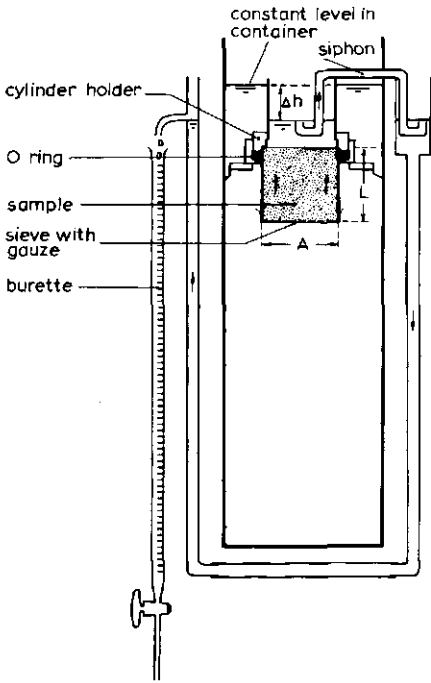


Fig.5. Position of Kopecky ring sample in the apparatus of Fig.4 for measuring hydraulic conductivity under a constant head (after WIT, 1967)

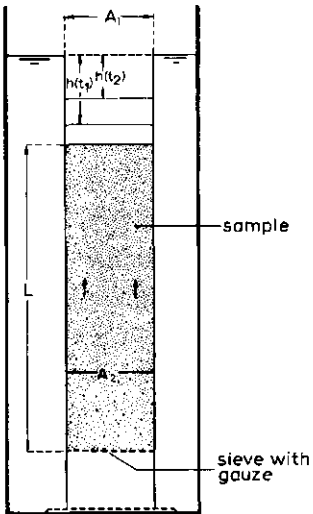


Fig.6. Position of sample in the apparatus of Fig.4 for measuring hydraulic conductivity under a falling head (after WIT, 1967)

Example 4

A soil sample is taken below the groundwater table in a zinc tube 6.2 cm in diameter and 30 cm long. After some soil is removed from the top and the bottom, a soil sample of 25 cm remains. The hydraulic conductivity is determined by the falling head method as illustrated in Fig.6 ($A_1 = A_2 = A$).

At time $t_1 = 0$ hours $h(t_1)$ measures 1.5 cm
At time $t_2 = 7$ hours $h(t_2)$ measures 0.9 cm

K is calculated by Eq.13 as

$$K = \frac{25}{7/24} 2.3 \log \left(\frac{1.5}{0.9} \right) = 43 \text{ cm/day} = 0.43 \text{ m/day}$$

24.3.3 K-VALUES OF SAMPLES TAKEN IN HORIZONTAL AND VERTICAL DIRECTIONS

Samples can be collected above the water table with the Kopecky ring in either a horizontal or a vertical direction. The measurements will then yield K values in horizontal or vertical direction, respectively. It is virtually impossible to collect samples in horizontal direction from deeper soil layers or from layers below the water table, but soil samples taken vertically can be measured for their hydraulic conductivity in horizontal direction by using the apparatus shown in Fig.7.

The base of a sample 30 cm long and 6 cm in diameter is sealed with clay and a plastic sheet. Two holes 1 cm in diameter are sucked into the top of the sample and on opposite sides of it. This can be done with a brass tube connected to a waterjet pump. Two tubes containing filters 10 cm long are then inserted into the holes. The sample is sealed at the top with a mixture of paraffin and vaseline, after which it is placed in water under vacuum to saturate it. The sample is then put into the hydraulic conductivity apparatus (Fig.4) and the constant head procedure is followed. The water flows horizontally through the sample from one filter to the other. The dimensions of sample and filter tubes are such that, for the pattern of streamlines and equipotential lines, the following relation holds

$$K = \frac{Q}{\Delta h L} \tag{14}$$

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where

- K = hydraulic conductivity (horizontal) (cm/day)
- Q = flow of water through sample (cm³/day)
- Δh = constant head loss (cm)
- L = length of filters (cm)

If the temperature and viscosity of the water used in the laboratory differ from that of the groundwater, the following correction is applied (see also Chap.6, Vol. I)

$$K_g = \frac{\eta_1}{\eta_g} K_1 \quad (15)$$

where

- K_g, K₁ = hydraulic conductivity (cm/sec) for groundwater and laboratory, respectively,
- η_g, η₁ = viscosity (poise) for groundwater and laboratory, respectively.

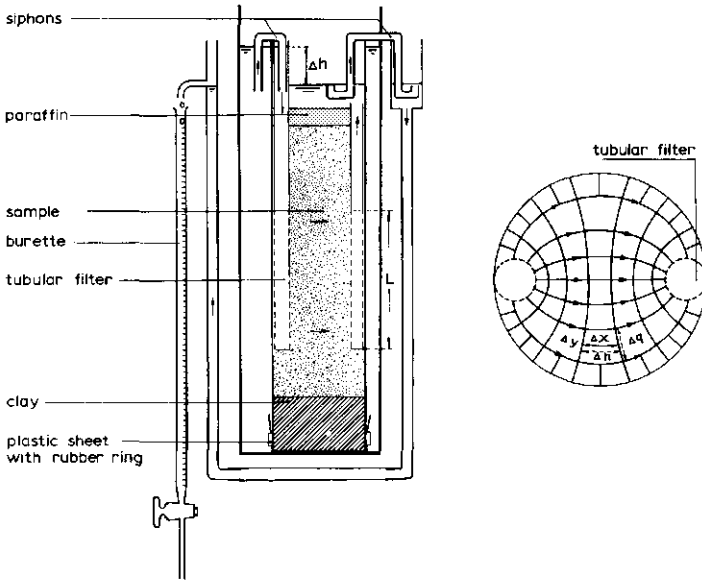


Fig.7. Measuring hydraulic conductivity of long core samples in a horizontal direction

A few examples of soil profiles with different hydraulic conductivities in horizontal and vertical directions are presented in Fig.8.

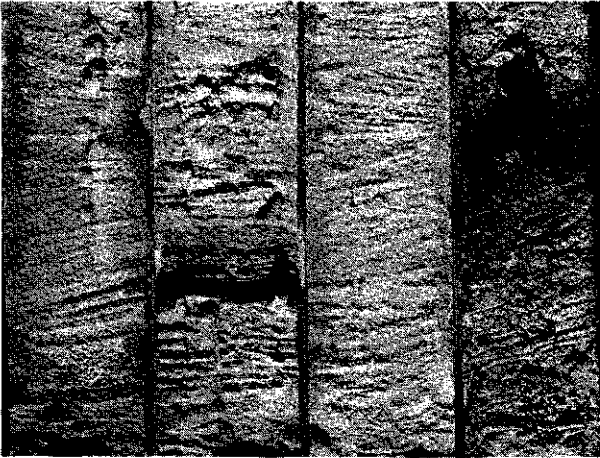


Fig.8. Core samples from laminated tidal flat deposits with different hydraulic conductivities in horizontal (K_h) and vertical (K_v) directions

$k_h=5.0$ m/day $k_h=5.5$ m/day $k_h=8.1$ m/day $k_h=5.0$ m/day
 $k_v=2.5$ m/day $k_v=0.9$ m/day $k_v=8.2$ m/day $k_v=1.4$ m/day

Discussion

The laboratory methods measure hydraulic conductivity in a horizontal or vertical direction in small, undisturbed soil samples, taken from above or below the water table. They therefore have a wide application. Compared with other methods, however, they have two inherent limitations. The first is that they are time-consuming, require adequate laboratory facilities, and demand careful sampling techniques. The second limitation is the small sample size, which gives rise to large sampling fluctuations (random errors) and means that many samples must be taken before one can arrive at a reliable evaluation. The small sample size will not constitute a serious limitation when one is investigating the hydraulic conductivity of thin layers. Nor will it be a disadvantage when very deep layers are being studied as these usually have a greater homogeneity than shallower layers.

24.4 FIELD METHODS

24.4.1 AUGER HOLE METHOD

Principle

The auger hole method can be used to measure hydraulic conductivity in situ below a water table. A detailed description of the procedure is given by VAN BEERS (1958). The principle of the method is as follows. A hole is bored into the soil with an auger to a certain depth below the water table. When the water in the hole reaches equilibrium with the groundwater, part of it is removed. The groundwater then

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begins to seep into the hole and the rate at which it rises is measured. The hydraulic conductivity of the soil is computed by a formula or graph describing the mutual relation between the rate of rise, the groundwater conditions, and the geometry of the hole. This method measures the average hydraulic conductivity of a soil column about 30 cm in radius and extending from the groundwater table to about 20 cm below the bottom of the hole, or to a relatively impermeable layer if it occurs within 20 cm of the bottom.

Simple and convenient measuring equipment has been developed in The Netherlands; it is illustrated in Fig.9. It consists of a tube, 60 cm long, the bottom end of which is fitted with a clack valve so that it can be used as a bailer. Extension pieces can be screwed to the top end of the tube. A float, a light-weight steel tape, and a standard are also part of the equipment. The standard is pressed into the soil up to a certain mark, so that the water level readings can be taken at a fixed height above the ground surface.

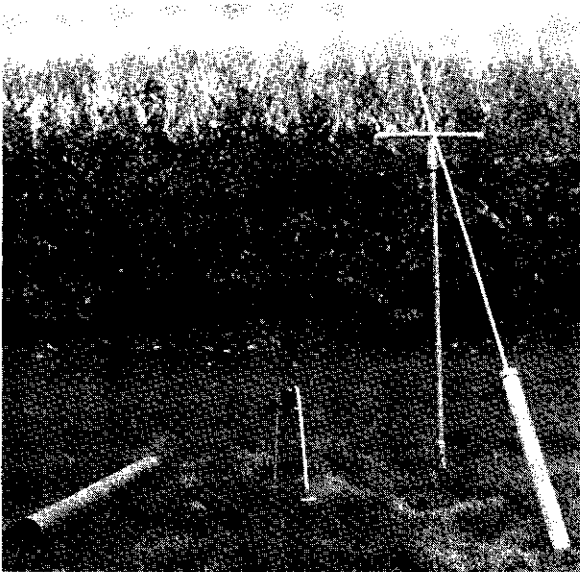


Fig.9. Equipment for the auger-hole method (after VAN BEERS, 1965)

The hole must be made with a minimum of disturbance to the soil. The open blade auger used in The Netherlands is very suitable for wet clay soils, whereas the closed posthole auger commonly used in the U.S.A. is excellent in dry soils. The depth of the holes depends on the nature, thickness, and sequence of soil layers, and on the depth at which one wishes to determine hydraulic conductivity. When the water in the hole is in equilibrium with the groundwater, the level is recorded. Water is then bailed out to lower the level in the hole by 20 to 40 cm ($h(\tau_1) = 20$ to 40 cm, Fig.10).

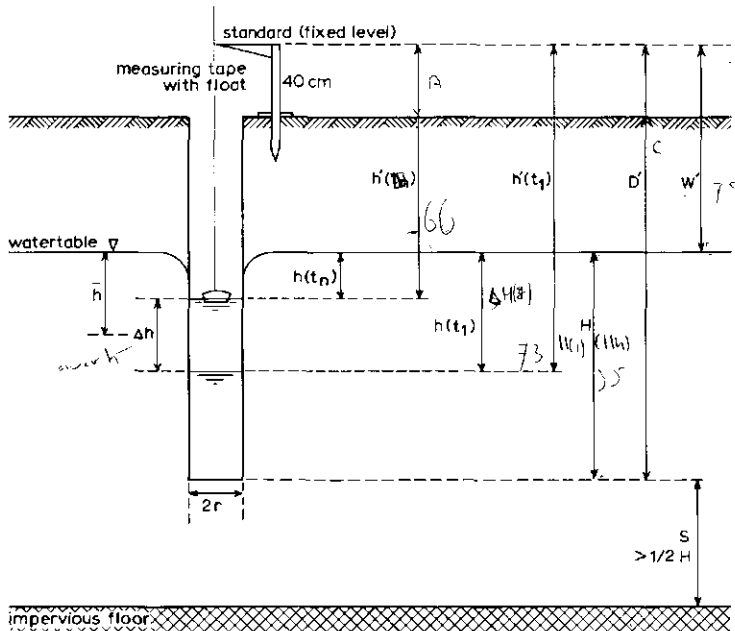


Fig.10. The auger-hole method. D' = depth of the auger hole below level of the standard; W' = depth of the water table below level of the standard; $H = D' - W'$ = depth of the auger hole below water table; $h'(t_1)$, $h'(t_n)$ = depth of the water table in the hole below standard level at the time of the first reading (t_1) and after some readings (t_n). Usually about 5 readings are taken; $\Delta h = h'(t_1) - h'(t_n) = h(t_1) - h(t_n)$ = the rise of water level in the hole during the time of measurements; $h = h(t_1) - \frac{1}{2}\Delta h$ = average head during the time of measurements; S = depth of impervious floor below the bottom of the hole; r = radius of the hole

Measurement of the rate of rise in the water level must begin immediately after bailing. Either the time for fixed intervals of rise, or the rise for fixed intervals of time can be recorded. The first technique requires the use of chronometers while the second, which is customary in The Netherlands, needs only a watch with a good second hand. Normally some 5 readings are taken, as these will give a reliable average value for the rate of rise and also provide a check against irregularities. The time interval at which water level readings are taken is usually from 5-30 seconds, depending on the hydraulic conductivity of the soil, and should correspond to a rise of about 1 cm in the water level. A good rule of thumb is that the rate of rise in mm/sec in an 8 cm ϕ hole to a depth of 70 cm below the water table approximately equals the K-value of the soil in m/day.

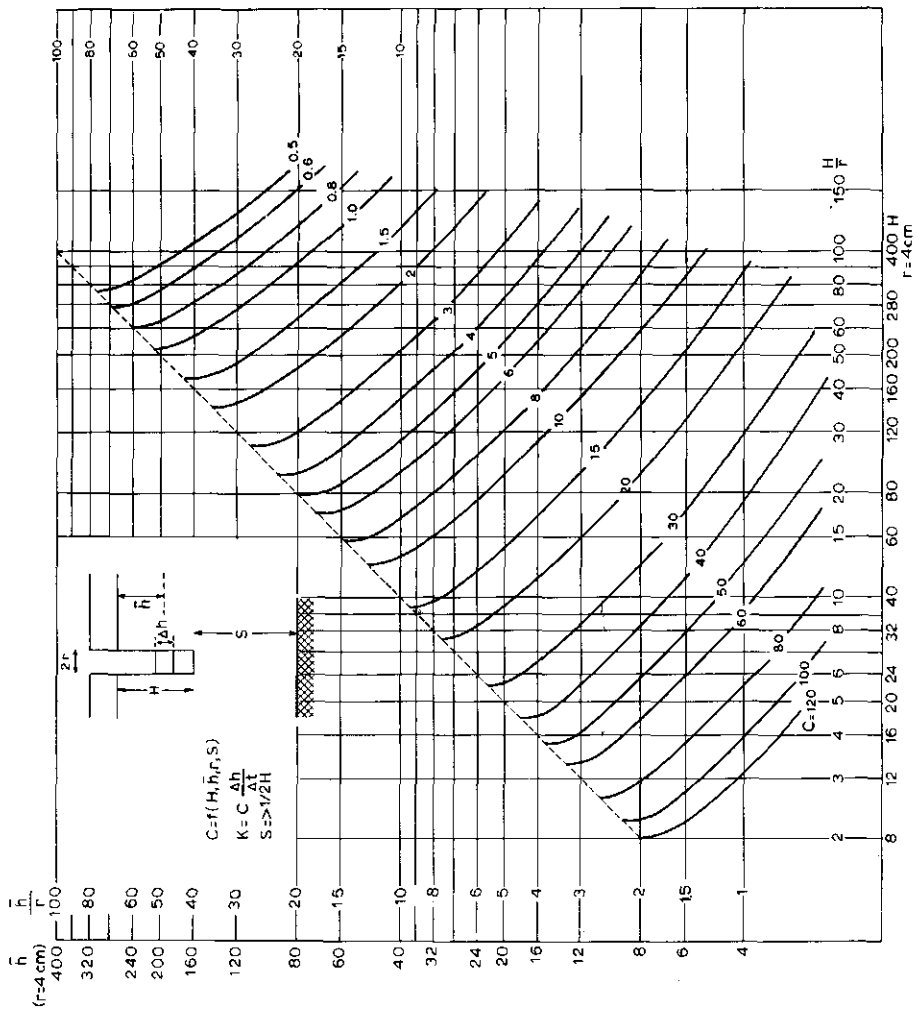


Fig.11. Nomograph for determination of C in auger-hole method (after ERNST, 1950)

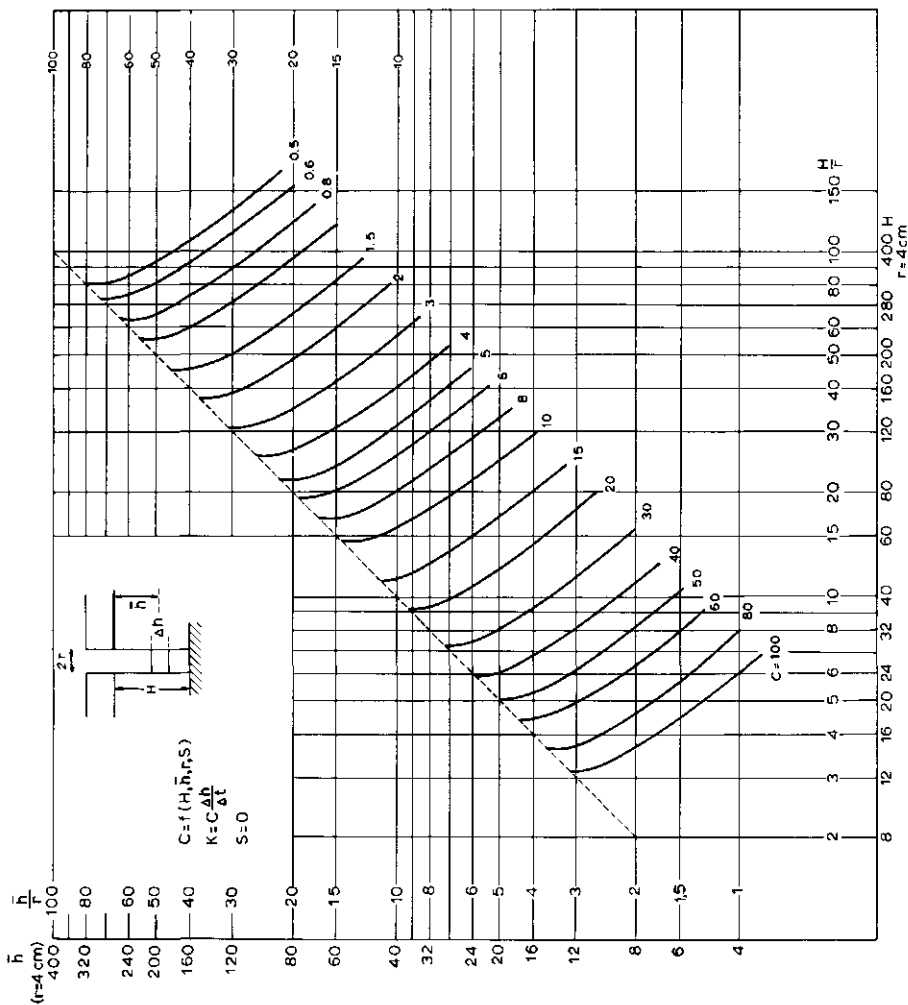


Fig. 12. Nomograph for determination of C in auger-hole method (after ERNST, 1950)

Care should be taken to complete the measurements before 25% of the volume of water removed from the hole has been replaced by inflowing groundwater. After that a considerable funnel shaped watertable develops around the top of the hole. This increases resistance to the flow around and into the hole. This effect is not accounted for in the formulas or flow charts developed for the auger hole method and consequently it should be checked that $\Delta h < \frac{1}{4} h(t_1)$ (Fig.10).

Calculations (the one layer problem)

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

$$K = C \frac{\Delta h}{\Delta t} \tag{16}$$

where

K = hydraulic conductivity (m/day)

C = geometry factor = $f(\bar{h}, H, r, S)$. (Figs.10,11, and 12)

$\frac{\Delta h}{\Delta t}$ = rate of rise of water level in the auger hole (cm/sec)

In Fig.11 C is given as a function of \bar{h}/r and H/r when $S > \frac{1}{4}H$, in Fig.12 a similar relation applies when $S = 0$. The use of these figures can be illustrated by the following example.

Example 5

After the readings have been taken, the reliability of the measurements should be checked. The Δh of each measurement is therefore computed to see whether the consecutive readings are reasonably consistent. If the value of Δh decreases gradually, the readings may be averaged up to $\Delta h = \frac{1}{4} h(t_1)$ or as in the above example, up to $\Delta h = 7$ to 8 cm. Both conditions are satisfied here and therefore K can be computed.

It often happens that Δh is relatively large for the first reading, due to water dripping along the walls of the hole directly after bailing. If this occurs, the first measurement should be discarded. Further inconsistency in Δh values may be caused by the float sticking to the wall or by the wind blowing the tape against the wall. Consistency can be improved by tapping the tape regularly.

No.:				Date:	
Location:				Technician:	
D' = 240 cm				r = 4 cm	
W' = 114 cm				S = > 1/2 H	
H = D' - W' = 126 cm					
i	t _i	h'(t _i)	Δh	Δt = t ₆ - t ₁ = 50 sec	
	second	cm	cm	Δh = h'(t ₁) - h'(t ₆) = 5.6 cm	
1	0	145.2	-	h(t ₁) = h'(t ₁) - W' = 145.2 - 114 = 31.2 cm	
2	10	144.0	1.2	check Δh < 1/4 h(t ₁) (5.6 < 7.8)	
3	20	142.8	1.2	h̄ = h(t ₁) - 1/4 Δh = 31.2 - 2.8 = 28.4 cm	
4	30	141.7	1.1		
5	40	140.6	1.1	H/r = 31.5	
6	50	139.6	1.1	h̄/r = 7.1 } C = 6.0 (read from Fig.11)	
		Δh = 5.6	5.6	Δh/Δt = 5.6/50 = 0.11	
				K = C Δh/Δt = 6.0 × 0.11 = 0.66 m/day	

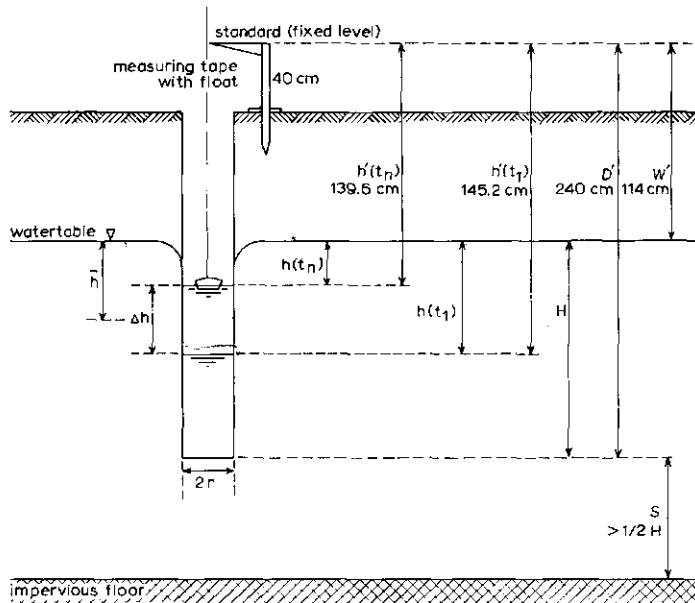


Fig.13. Example of measurements for auger-hole method

Calculations (the two layer problem)

If a soil profile consists of two layers of different hydraulic conductivity, K_1 in the upper layer and K_2 in the lower layer, these values for the separate layers can be determined if the water table is well within the upper layer. Two successive measurements are made in one hole, the second measurement being taken after the original hole has been deepened. A hole is bored to at least 40 cm below the water table, but should not extend further than 20 cm above the lower layer. The deepened bore hole must reach at least 50 cm into the lower layer. The measurement in the shallow hole gives K_1 , the hydraulic conductivity of the upper layer, in the same way as for a homogeneous profile.

$$K_1 = C_1 (\Delta h / \Delta t)_1 \quad (17)$$

where

$$C_1 = f(\bar{h}_1, H_1, r, S_1 > \frac{1}{2}H_1)$$

The rate at which the water in the deep hole rises is thought to be the result of two components (Fig.14)

- inflow from the upper layer with hydraulic conductivity K_1 only, the lower layer being considered impermeable;
- inflow from the lower layer which is considered to consist of inflow from the whole profile with hydraulic conductivity K_2 minus inflow from the upper layer, also with hydraulic conductivity K_2 , the lower layer being considered impermeable.

Hence
$$(\Delta h / \Delta t)_2 = K_1 / C_o + K_2 / C_2 - K_2 / C_o \quad (18)$$

where

$$C_o = f(\bar{h}_2, H_o, r, S_1 = 0) \quad (\text{Fig.12})$$

$$C_2 = f(\bar{h}_2, H_2, r, S_2 > \frac{1}{2}H_2) \quad (\text{Fig.11})$$

Rearranging gives

$$K_2 = \frac{C_o (\Delta h / \Delta t)_2 - K_1}{C_o / C_2 - 1} \quad (19)$$

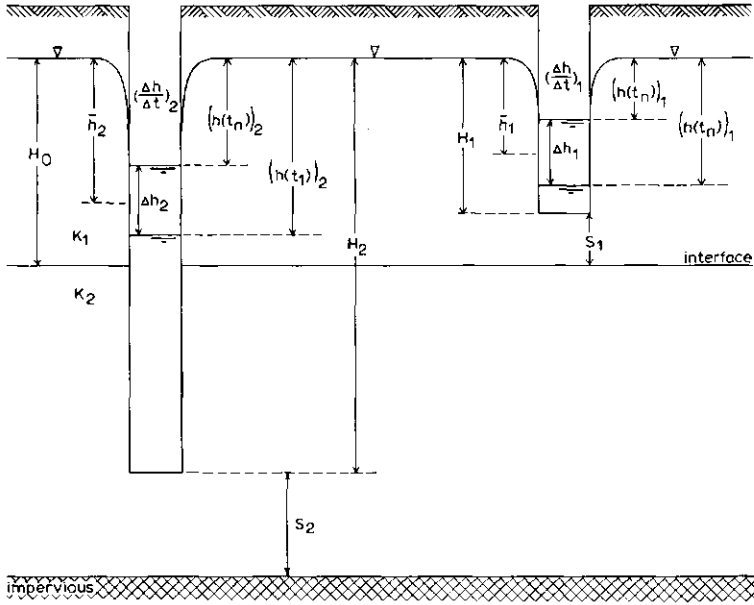


Fig.14. The auger-hole method for the two-layer problem

Example 6

$$\begin{array}{l}
 (\Delta h/\Delta t)_1 = 0.16 \\
 H_1 = 70 \text{ cm} \\
 \bar{h}_1 = 30 \text{ cm} \\
 S_1 > \frac{1}{2} H \\
 r = 4 \text{ cm}
 \end{array}
 \left. \vphantom{\begin{array}{l} (\Delta h/\Delta t)_1 \\ H_1 \\ \bar{h}_1 \\ S_1 \\ r \end{array}} \right\} C_1 = 9.4 \quad (\text{Fig.11})$$

$$K_1 = C_1 (\Delta h/\Delta t)_1 = 9.4 \times 0.16 = 1.5 \text{ m/day}$$

$$\begin{array}{l}
 (\Delta h/\Delta t)_2 = 0.26 \\
 H_2 = 150 \text{ cm} \\
 \bar{h}_2 = 40 \text{ cm} \\
 S_2 > \frac{1}{2} H
 \end{array}
 \left. \vphantom{\begin{array}{l} (\Delta h/\Delta t)_2 \\ H_2 \\ \bar{h}_2 \\ S_2 \end{array}} \right\} C_2 = 3.9 \quad (\text{Fig.11})$$

$$\begin{array}{l}
 H_0 = 100 \text{ cm} \\
 \bar{h}_2 = 40 \text{ cm} \\
 S_1 = 0
 \end{array}
 \left. \vphantom{\begin{array}{l} H_0 \\ \bar{h}_2 \\ S_1 \end{array}} \right\} C_0 = 6.3 \quad (\text{Fig.12})$$

$$K_2 = \frac{C_0 (\Delta h/\Delta t)_2 - K_1}{C_0/C_2 - 1} = \frac{6.3 \times 0.26 - 1.5}{6.3/3.9 - 1} = 0.22 \text{ m/day}$$

Discussion

Variations in the results of field investigations are less than those obtained in the laboratory because in the field a larger soil body is measured. The error of the determination is believed to be of the order of 10-20%. An accompanying soil profile description is indispensable for further evaluation of the terrain variability (Sect.5.2).

In stable soils of low hydraulic conductivity, it may be necessary to extend the augerhole 1 m or more below the water table, and to bail out more than 80 cm of water to create a measurable rate of rise. In heavy soils the action of the auger may destroy the soil structure in the vicinity of the wall of the hole. It will then be necessary to flush the soil and reopen the closed soil pores by bailing the water out several times. Repeated measurements will reveal that the hydraulic conductivity gradually improves with time.

In unstable soils, or in soils with a very high hydraulic conductivity, auger holes deeper than 40 cm or bailings of more than 20 cm of water may produce a rate of rise too high to be measured. A cave in unstable soils can be prevented by inserting a filter into the hole. It is advisable to measure the rate of rise immediately after making the hole, in which case no bailing is required (direct method). Repeated measurements in an unstable hole, where mud may collect on the bottom, will reveal a gradual decrease in hydraulic conductivity.

Figs.11 and 12 only differ significantly in the range of small H (depth of the borehole) values. If one is uncertain of the conductivity of the layers below the bottom of the hole, Fig.11 may be used ($S > \frac{1}{2}H$), thus ensuring a conservative estimate of K.

The simplicity of the augerhole method, the fairly large soil body for which K is measured (about 0.05 to 0.3 m³), and the presence of relatively high water tables explain the frequent use of this method in The Netherlands.

24.4.2 PIEZOMETER METHOD

Principle

The principle of this method resembles that of the augerhole method, except that a tube is inserted into the hole to leave a small cavity at the bottom. The method is therefore often referred to as the pipe cavity method.

The principle is as follows (Fig.15). A hole is augered into the soil to the depth below the water table at which one wishes to measure the hydraulic conductivity. A pipe is inserted into the hole, having a small unprotected cavity at the bottom. Care should be taken that no leakage occurs along the wall of the hole. After the water level in the pipe has reached equilibrium with the groundwater, the water in the pipe is removed. Due to a pressure difference, water will flow from the surrounding soil into the uncased cavity, causing the water level in the pipe to rise. The rate of rise is measured and the K value is calculated with a suitable formula describing the relation between the rate of rise, the flow conditions and the K value of the soil. The equipment consists of an auger, a pipe, a pump or bailer, and a water level recorder.

Calculations

The hydraulic conductivity is computed with the following formula (LUTHIN and KIRKHAM, 1949)

$$K = \frac{\pi r_p^2}{C(t_2 - t_1)} \ln \frac{h(t_2)}{h(t_1)} \quad (20)$$

where

K = hydraulic conductivity (cm/sec)

r_p = inside radius of the pipe (cm)

$h(t_2)$, $h(t_1)$ = depth of water level (cm) in the pipe below equilibrium water level at time t_2 and t_1 respectively

$t_2 - t_1$ = time interval of measurement (sec)

C = $f(H, r_c, l, S)$, geometry factor

The above formula is similar to the formula used in the falling head laboratory method (Sect.3.2).

The geometry factor C has been determined with an electric analogue, whereas in the auger hole method it was determined by relaxation. The result of the electric analogue experiments is given in Fig.16.

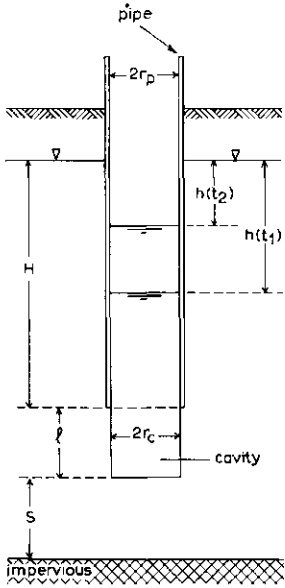


Fig.15. The piezometer method

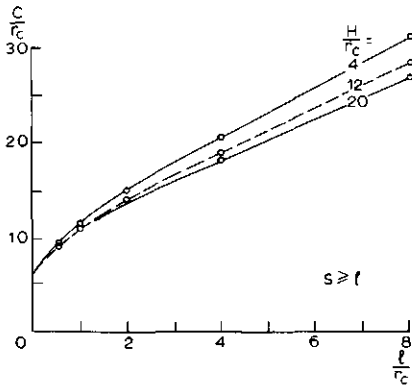


Fig.16. Nomograph for the determination of C in the piezometer method according to SMILES and YOUNGS (1965)

Example 7

A piezometer with an inside radius of 5 cm is driven into the soil to 1 m below the water table. Below the piezometer a cavity 25 cm deep and 5 cm in radius is made. After equilibrium is reached, water is pumped out of the piezometer. The following measurements of the rate of rise are made

i	t_i (sec)	$h(t_i)$ (cm)	$\frac{\ln h(t_1)/h(t_i)}{t_i - t_1}$	
1	0	70		
2	20	65	3.68	10^{-3}
3	40	60	3.85	10^{-3}
4	60	56	3.72	10^{-3}
5	80	52	3.71	10^{-3}

The ratio

$$\frac{\ln h(t_1)/h(t_i)}{t_i - t_1}$$

is first calculated to check the consistency of the data series. The factor C is determined from the nomograph Fig.16 with $1/r_c = 5$ and $H/r_c = 20$, giving $C = 100$. The K-value found from Eq.20 will be in cm/sec, and needs to be multiplied by 864 to convert it into m/day.

$$K = \frac{864 \pi r^2 P}{C(t_i - t_1)} \ln \frac{h(t_1)}{h(t_i)} = \frac{864 \times \pi \times 5^2 \times 3.71 \times 10^{-3}}{100} = 2.5 \text{ m/day}$$

Discussion

The piezometer method is highly suitable for determining the hydraulic conductivity of individual soil layers or of layers at great depths. Of the hydraulic head, 80% is dissipated in the soil within a distance of one cavity length from the cavity, and about 90% within two cavity lengths. This method therefore measures the hydraulic conductivity of the soil within a radius of one or two cavity lengths from the cavity (0.01 - 0.05 m³), in a horizontal direction.

It may be necessary to support the cavity with a filter in unstable soil. The smaller inflow surface of the cavity compared with that provided by the auger hole method heightens the entrance velocity of the water. This increases the risk of collapse or deformation of the cavity, and might lead to considerable error. As with the augerhole method, it may be necessary to flush the cavity several times in stable clay soils if its walls have been sealed by the action of the auger.

24.4.3 PUMPED BOREHOLE METHOD

Principle

The principle of the method is as follows. A hole is bored into the soil to below the water table. Water is then pumped from it at a constant rate. When the water level in the hole has reached equilibrium, the K value is calculated with a suitable formula expressing the relation between the stabilized water level in the borehole, the hydraulic conductivity of the surrounding soil, and the flow conditions.

The equipment consists of an auger, a pump, and a waterlevel recorder (Chap.25, Vol.II).

Calculation

ZANGAR (1953) has given an analytical solution to the flow problem of a well penetrating less than 20% into an aquifer. The expression for the hydraulic conductivity is as follows (Fig.17)

$$K = \frac{Q}{C L r} \tag{21}$$

where

K = hydraulic conductivity (m/day)

Q = steady rate of pumping (m^3/day)

r = radius of the borehole (m)

$$L = \frac{H^2 - h^2}{H}$$

H = depth of the bottom of the borehole below the initial groundwater table (m)

h = stabilized height of the water level above the bottom of the borehole at equilibrium (m)

C = f(h,r), geometry factor (see Fig.18)

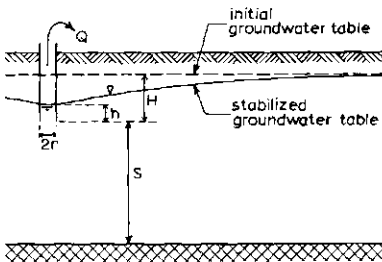


Fig.17. The pumped borehole method

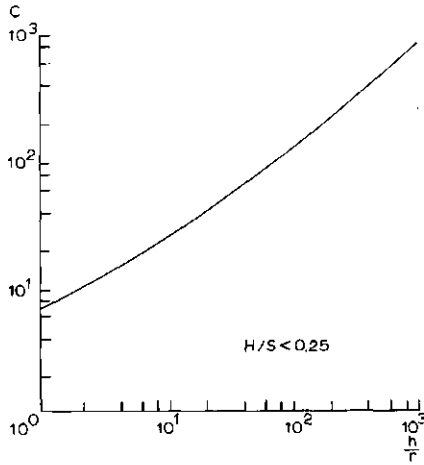


Fig.18. Nomograph for the determination of C for the pumped borehole method (after ZANGAR, 1953)

Example 8

The water table in a pervious layer 15 m thick is 1 m below the surface. A bore hole 0.10 m in radius is dug to 3 m below the surface; hence $H = 3 - 1 = 2$ m and $S = 15 - 3 = 12$ m. Water is withdrawn at a rate of 34 l/min (=49 m³/day). After some time the water level in the hole stabilizes at 0.50 m above the bottom of the borehole ($h = 0.50$ m).

Hence, according to Eq.21,

$$L = \frac{H^2 - h^2}{2H} = \frac{2^2 - 0.5^2}{2 \times 2} = 0.94 \text{ m}$$

Further

$$\frac{h}{r} = \frac{0.5}{0.1} = 5$$

and thus from Fig.18: $C = 18$.

Substituting the numerical values into Eq.21 produces

$$K = \frac{Q}{C L r} = \frac{49}{18 \times 0.94 \times 0.10} = 29 \text{ m/day}$$

Discussion

The pumped borehole method is best applied to deep homogeneous phreatic aquifers of high hydraulic conductivity. The aquifer should be deep enough to satisfy the condition that no more than 20% penetration occurs ($H/D = 0.25$), and the hydraulic conductivity should be high enough to guarantee measurable pumping rates and equilibrium water levels. This indicates the advantage of the pumped borehole in highly pervious aquifers since under these conditions, the augerhole method, and to a certain extent the piezometer method, may produce rates of rise of the water level too high to be measured accurately. The pumped borehole differs from most other pumping tests in that an additional installation of piezometers is not required. The K-value of the soil is measured in a direction intermediate between horizontal and vertical and is representative of a large soil body of several m^3 .

24.4.4 DOUBLE TUBE METHOD

Principle

The principle of the double tube method is illustrated in Fig.19. In an unsaturated soil, two concentric tubes are placed at the depth where one wishes to measure the hydraulic conductivity. First, the soil below and around the tubes is saturated by infiltration; note that the infiltration rate must be constant. The tubes are then filled with water and the rate of fall of the levels in both outer and inner tube is measured. The tubes are then refilled, the water level in the outer tube now being kept constant, and the rate of fall in the inner tube being recorded. Because of a gradually increasing difference between water levels in the outer and inner tubes, the infiltration from the inner tube will be counteracted by a gradually increasing hydraulic head from the outer tube. This reduces the rate of fall in the inner tube, as compared to the rate of fall occurring when the equal level condition is imposed. The hydraulic conductivity can be computed with a formula expressing the relation between the difference in fall under equal level and constant outer level conditions, the flow geometry, and the hydraulic conductivity. The appropriate equipment for the technique, as developed by BOUWER (1961, 1962), is schematically described in Fig.20. It consists of a water reservoir connected by pipes and valves to the inner and outer tubes in such a way that the conditions of equal level fall and constant outer level can be imposed. Also required are a large posthole auger and cleaning equipment to return the disturbed structure of the bottom of the hole to its original condition.

Calculations

The hydraulic conductivity is computed with the following equation

$$K = \frac{r_p^2}{r_c C} \frac{\Delta h}{t \int_0^t h_c(t) dt} \quad (22)$$

where (see Fig.19)

- K = hydraulic conductivity (mm/min)
- r_p = radius of standpipe of inner tube (mm)
- r_c = radius of inner tube (mm)
- $\Delta h = h_e(t) - h_c(t)$ (mm)
- $h_e(t)$ = drop of water level in inner tube under equal level condition in a time interval of t minutes (mm)
- $h_c(t)$ = drop of water level in inner tube under constant outer level condition in a time interval of t minutes (mm)
- C = geometry factor

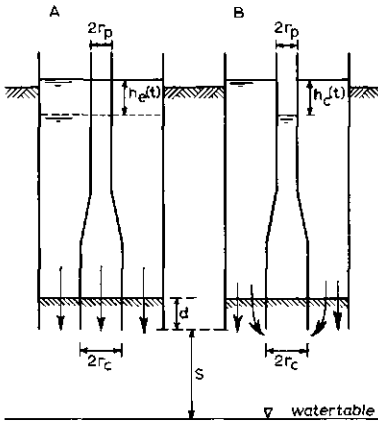


Fig.19. Principle of measuring hydraulic conductivity in non-saturated soil with the double tube method. A: Infiltration from inner and outer tubes under falling water levels, which are kept equal in both tubes. B: Infiltration from inner and outer tubes under falling water level in inner tube and constant water level in outer tube

The geometry factor C has been developed by BOUWER (1961) from analogue electrical resistance networks. It appears that C is virtually independent of S (Fig.20) if $S/r_c > 3$. Under this condition, C, as a function of d/r_c only, can be found from Table 4.

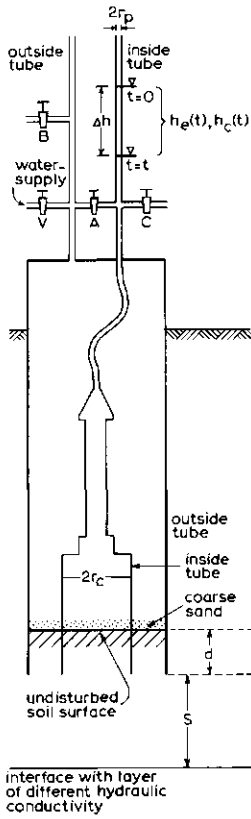


Fig.20. Equipment for the double tube method

Table 4. The relation between the geometry factor C and d/r_c for the double tube method (modified after BOUWER)

d/r_c	C
0.10	2.2
0.15	1.9
0.20	1.6
0.30	1.4
0.40	1.2
0.60	1.0
0.80	0.8
1.00	0.7

Example 9

The double tube method is applied in an auger hole, the bottom of which is 1 m below the soil surface yet above a water table. The inner tube 62.5 mm in radius penetrates 10 mm into the bottom of the auger hole. The radius of the standpipe, in which the observations are made, is 12.5 mm. The results of experiments are presented in Fig.21.

From the data given in this figure, it follows that

$$r_p = 12.5, r_c = 62.5 \text{ mm, and } d = 10 \text{ mm}$$

Consulting Table 4 one finds that for $d/r_c = 10/62.5 = 0.16$, C is 1.8.

Hence, at $t = 30 \text{ min}$, $\Delta h = 40 \text{ mm}$, and $h_c(t) = 20 \text{ mm}$. The integral value of Eq.22 is approximated by a triangle with an area

$$\frac{1}{2} \times t \times h_c(t) = \frac{1}{2} \times 30 \times 20 = 300$$

Inserting the above information into Eq.22 produces

$$K = \frac{r_p^2}{r_c C} \frac{\Delta h}{t \int_0^t h_c(t) dt} = \frac{(12.5)^2}{62.5 \times 1.8} \frac{40}{300} = 0.18 \text{ mm/min} = 0.26 \text{ m/day}$$

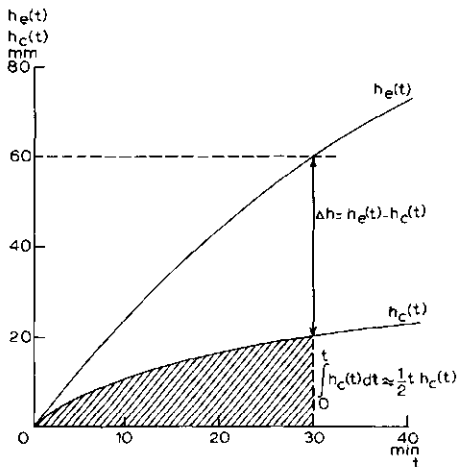


Fig.21. Graphical analysis of measurements with double-tube method

Discussion

In areas where irrigation is to be introduced, water losses may raise the water table to such an extent that drainage becomes necessary. If this problem is anticipated a drainage lay-out should be designed. Until recently, the hydraulic conductivity of soil layers above a water table could only be determined by laboratory methods. The double tube technique is a newly developed field method, the value of which is unfortunately not yet fully appreciated due to a lack of experience with it. As the double tube method makes use of an infiltration surface, the conservation of the original soil structure is of utmost importance. The application of a thin layer of sand to the infiltration surface is advisable in unstable soils. The tubes must be installed very carefully as otherwise leakage will occur along the tube wall. The application of Eq.22 is conditional upon the infiltration rates remaining constant during the test. It is therefore advisable to repeat the equal level infiltration test several times before the experiment and once afterwards to check on the constancy. Further, the constant outer level test should be done as quickly as possible. The volume of water needed for one test depends on the diameter of the tubes and on the hydraulic conductivity of the soil, but under average conditions at least 500 l will be required. The heavy apparatus together with the precautions that must be taken makes the double tube method fairly involved and time-consuming. LINSEN (1969) experimented with this method and concluded that the double tube method could not be recommended as a quick and efficient technique.

24.4.5 INFILTRMETER METHOD

Principle

The cylinder infiltrometer, used in irrigation studies to determine the infiltration rate and cumulative infiltration, can be used at successive depths in a soil pit to study the differences in the hydraulic conductivity of the various layers. The principle of this method is the same as that of the double tube method.

First one infiltrates water around the infiltrometer till the soil is saturated. The infiltrometer is then filled with water, and the rate at which the water level falls is measured. After some time the infiltration rate stabilizes and approximates the hydraulic conductivity K (Fig.22).

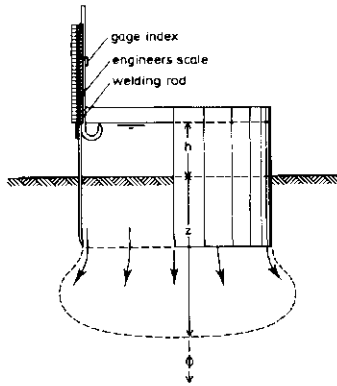


Fig.22. The infiltrometer

Calculations

The infiltration rate of water in an unsaturated soil measured by a cylinder infiltrometer may be expressed in terms of Darcy's law as

$$v = K_T \frac{\phi + z + h}{z} \tag{23}$$

where

- v = infiltration rate (cm/sec)
- K_T = hydraulic conductivity of the transmission zone (cm/sec)
- ϕ = suction at the bottom of the transmission zone (cm)
- z = depth of the transmission zone below the infiltrometer (cm)
- h = height of water in the infiltrometer (cm)

The influence of ϕ and h relative to z diminishes as the depth of the transmission zone and the moisture content of the soil increase. So the hydraulic gradient

$$\left(\frac{\phi + z + h}{z} \right)$$

tends towards 1 in the case of a deep uniform soil profile and the infiltration rate becomes constant, attaining what is known as the basic infiltration rate.

In that case we may write

$$v \approx K_T \tag{24}$$

Hydraulic conductivity

For wet, medium, and heavy textured soils in which the hydraulic conductivity of the transmission zone is approximately the same as in the saturated zone, we get

$$v \approx K_T \approx K \quad (25)$$

When using the infiltrometer for hydraulic conductivity studies - more specifically for studies on the basic infiltration rate in moist soils - the measurements should extend for a period long enough to permit a constant infiltration rate to be obtained. This may take quite some time in dry clay soils, because a decrease in the infiltration rate may be caused by the decrease in the hydraulic gradient as well as by a change in hydraulic conductivity due to swelling. The value obtained in a layered soil only applies to the depth of soil penetrated by the infiltrometer, since lateral flow will occur below if the hydraulic conductivity of the underlying layer is low. If it is possible to determine the distance over which the lateral flow extends in the underlying layers by estimating the change in moisture content, the infiltration rate in the underlying layer can be calculated. This can be done by taking the ratio between the surface of the infiltrometer and the surface over which lateral flow occurs in the underlying layer and multiplying it by the infiltration rate in the infiltrometer. Figure 23 shows an example of lateral flow below an infiltrometer installed at the soil surface to a depth of 5 cm on a silty clay loam with a ploughed layer of 20 to 40 cm. In the situation of the example, the intake rate at a depth of 25 cm would be $(37/77)^2$ times the intake rate near to the bottom of the infiltrometer. At a depth of 65 cm the ratio is $(37/117)^2$.

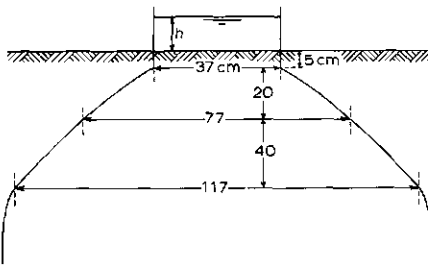


Fig.23. Lateral flow below infiltrometer

Discussion

The cylinder infiltrometer is suitable for determining the intake characteristics and hydraulic conductivities of irrigated soils. The results of this method are not very accurate, but can be regarded as a fair approximation of the K-value. The method is practical and suitable for large-scale surveys.

24.4.6 INVERSED AUGER HOLE METHOD

Principle

The auger hole test above the water table, described in French literature as the Porchet method, consists of boring a hole to a given depth, filling it with water, and measuring the rate of fall of the water level.

Calculations

The calculation is carried out as follows (Fig.24)

$$A(t_i) = 2\pi r h(t_i) + \pi r^2 \quad (26)$$

where

$A(t_i)$ = surface over which water infiltrates into the soil at time t_i (cm^2)

r = radius of the auger hole (cm)

$h(t_i)$ = water level in the hole at time t_i (cm)

Supposing that the hydraulic gradient is approximately 1, we may write according to Darcy's law

$$Q(t_i) = KA(t_i) = 2K\pi r (h(t_i) + r/2) \quad (27)$$

If during the time interval dt the water level falls over a distance dh , the quantity of water infiltrated into the soil equals

$$Q(t_i) = -\pi r^2 \frac{dh}{dt} \quad (28)$$

Substituting Eq.27 into Eq.28 gives

$$2K\pi r (h(t_i) + r/2) = -\pi r^2 \frac{dh}{dt} \quad (29)$$

Integration between the limits

$$t_i = t_1, h(t_i) = h(t_1) \quad \text{and}$$

$$t_i = t_n, h(t_i) = h(t_n) \quad \text{gives}$$

$$\frac{2K}{r} (t_n - t_1) = \ln (h(t_1) + r/2) - \ln (h(t_n) + r/2) \quad (30)$$

Changing to common logarithms and rearranging gives

$$K = 1.15 r \frac{\log (h(t_1) + r/2) - \log (h(t_n) + r/2)}{t_n - t_1} = 1.15 r \tan \alpha \quad (31)$$

By plotting $(h(t_i) + r/2)$ against t_i on semilogarithmic paper we obtain a straight line with a tangent α .

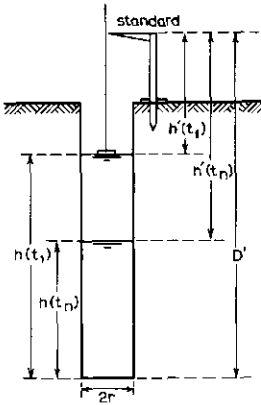


Fig.24. Inversed augerhole method

Example 10

$r = 4 \text{ cm}$
 $D' = 90 \text{ cm}$

i	t_i	$h'(t_i)$	$h(t_i)$	$h(t_i)+r/2$	t_i	$h'(t_i)$	$h(t_i)$	$h(t_i)+r/2$
	sec	cm	cm	cm	sec	cm	cm	cm
1	0	73	17	19	0	71	19	21
2	40	74	16	18	140	72	18	20
3	80	75	15	17	300	73	17	19
4	150	76	14	16	500	74	16	18
5	250	77	13	15	650	75	15	17
6	350	78	12	14	900	76	14	16
7	550	79	11	13	1090	77	13	15
8	750	80	10	12	1300	73	12	14
9	975	81	9	11	1520	79	11	13

$$\tan \alpha_1 = \frac{2.0}{10} \times \frac{1}{1200} \text{ sec}^{-1}$$

$$K = 1.15 \times 4 \times 0.000167 \text{ cm/sec}$$

$$= 0.66 \text{ m/day}$$

$$\tan \alpha_2 = \frac{2.7}{10} \times \frac{1}{2000} \text{ sec}^{-1}$$

$$K = 1.15 \times 4 \times 0.000135 \text{ cm/sec}$$

$$= 0.54 \text{ m/day}$$

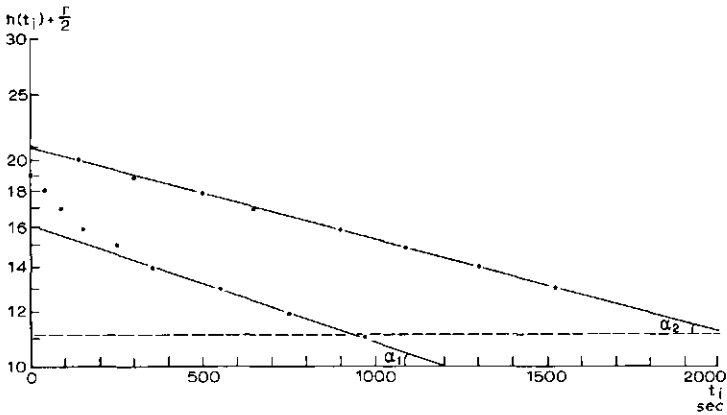


Fig.25. Plots of $h(t_i) + r/2$ versus t_i in the calculation of K

Discussion

In general measurement should be made 1 to 3 times in loam and clay soils, depending upon the moisture content of the soil and its hydraulic conductivity. It may be necessary to repeat the measurements 3 to 6 times on sandy soils. By gradually deepening the auger hole and filling it with water over the corresponding depth, the hydraulic conductivity of successive layers can be measured in the same hole.

The advantage of this method over the infiltrometer test lies in the difference between digging soil pits and boring auger holes. The infiltrometer test in soil pits is often the first phase of investigation in an area, while the inversed auger hole test can be applied in a later phase to measure the hydraulic conductivity at a large number of sites.

Remark on the methods described in Sects. 4.4, 4.5, and 4.6

When applying the above methods for measuring hydraulic conductivity in soil layers above the water table, one should always keep in mind that due to the swelling properties of the soil, the values obtained may differ from those of the saturated hydraulic conductivity. If this change of structure is significant, it has to be taken in consideration when the measured K is being evaluated.

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SURVEYS AND INVESTIGATIONS

25. DERIVING AQUIFER CHARACTERISTICS FROM PUMPING TESTS

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PURPOSE AND SCOPE

The procedure followed in performing pumping tests in unconfined and semi-confined aquifers is described, together with some methods of analyzing the data obtained from such tests.

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25.1 GENERAL SET-UP

The pumping test is a method widely used to determine the hydraulic characteristics of an aquifer. The basic procedure is simple: for a fairly long time and at a specified rate, water is pumped from a well which has a screen in the aquifer being tested. The effect of the pumping on the water table or the piezometric surface is measured both at the face of the well and in a number of observation wells in the vicinity. The hydraulic characteristics of the aquifer can then be found by substituting, in an appropriate well-flow formula, the drawdown measured in these observation wells, their distances from the pumped well, and the well discharge. In reality, a pumping test is much more difficult to conduct than might appear from this brief description. As will be shown below, if a pumping test is to yield reliable information, certain conditions must be satisfied and a certain skill is required.

In most regional groundwater investigations, the high costs of pumping tests and the usually limited funds available for this purpose normally allow only a small number of such tests to be conducted. It is sometimes possible to perform a pumping test on an existing well, although conditions may not be as ideal as when wells have been specially drilled for the purpose. In what follows it will be assumed that the pumped well and the observation wells have been specially drilled for the test.

25.1.1 SELECTION OF A PUMPING TEST SITE

When the test site is being selected, the following conditions should be satisfied:

- the hydrogeological conditions should be representative of the area under consideration, or at least a major part of it;
- manpower and equipment should be able to reach the site easily;
- it must be possible to discharge the pumped water outside the area affected by the pumping, thus preventing it from re-entering the aquifer;
- the natural gradient of the water table or piezometric surface should be small.

25.1.2 DISCHARGING WELL

The discharging (or pumped) well is a bored or drilled hole of sufficient size and depth, inside which a well screen is installed in the aquifer's most permeable layers and connected with a tube to the ground surface. The diameter of the well,

generally between 10 and 30 cm, depends on the type of pump that will be used, while the type of pump depends on the desired discharge rate and the allowable maximum pumping lift (Fig.1).

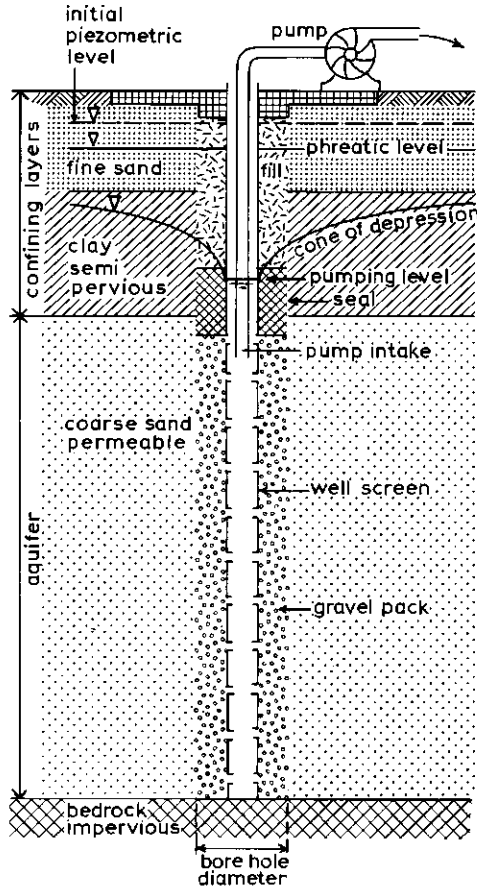


Fig.1. Pumped well in a semi-confined aquifer

Aquifer characteristics

If possible, the length of the well screen should equal the total saturated thickness of the aquifer (or at least 90% of it), so that when the well is pumped, the flow towards it will be essentially horizontal. Such a well is called a fully penetrating well. Its advantage is that the drawdown data to be analyzed later need not be corrected.

If the aquifer is thick, well screens shorter than the aquifer thickness are sometimes installed. In such partially penetrating wells, vertical flow components will influence the drawdown measurements within a radial distance from the well approximately equal to the thickness of the aquifer (Fig.2). Prior to their analysis the drawdown data must be corrected to eliminate these vertical flow components.

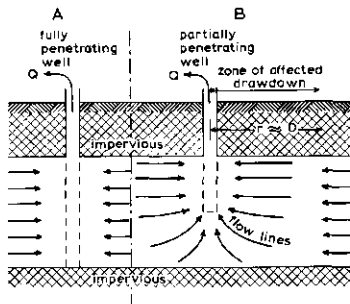


Fig.2. Schematic cross-section through (A) a fully and (B) a partially penetrating well

In aquifers made up of fine uniform sand or thin alternating layers of fine, medium, and coarse sands, the permeability of the zone immediately surrounding the well screen should be improved by removing the aquifer material and replacing it with artificially graded coarse material (gravel packing). This gravel pack retains the fine particles still present at some distance from the well and allows the passage of groundwater with a smaller loss of head.

25.1.3 WELL DEVELOPMENT

After the well has been completely constructed it is not yet ready for use. The first and most important step to be taken in bringing the well to its full capacity and ensuring safe and trouble-free operation is to develop the well by removing the finer particles from the formation that transmits the water towards the well. Eventually, the well should operate sand-free and at maximum capacity, with a drawdown not exceeding the permissible maximum.

Different techniques of well development can be applied, depending on whether the hole was drilled by the rotary or the percussion method. If the well was drilled

by the percussion method and no drilling mud was used, it can best be developed by repeatedly reversing the flow towards the well. This will break up the bridges of particles formed over the slots of the screen. These flow reversals can be obtained in different ways: by pouring water into the well and bailing it out, by alternately starting the pump and shutting it down, by pumping the well and jetting with water under high pressure. The most effective method of development is to use surge plungers and compressed air. If the wall of the hole has been severely clogged, the well capacity will be extremely low at the beginning. In such cases the well should be pumped at gradually increasing capacities before the surge plunger is used, or surging should start very slowly and speed up gradually as work progresses. If the wall of the hole is not severely clogged and an artificial gravel pack has been placed, development of the well can generally be restricted to de-sanding by pumping the well with gradually increasing capacities. Pumping should start at a rate equal to 20% of the permanent capacity and be continued till the water has become clear. The rate of pumping is then increased to 40% and the process repeated. This procedure should be continued up to 150% of the permanent capacity.

If drilling mud has been used to make the hole, polyphosphates will be required to disperse the clay particles of the mud cake.

For further details on well development, see JOHNSON (1966) and HUISMAN (1972). While the pumped well is being developed, the reaction of the water levels in the observation wells and piezometers should be checked. It is recommended that some development pumping also be done on these wells and piezometers, especially if mud was used for their construction.

25.1.4 TESTING OF THE PUMPED WELL

After the pumped well has been developed, a test for yield and drawdown is necessary to determine the capacity of the well and to select the proper type of pump that will eventually be installed. For this purpose the well is pumped in stages of increased capacity and the drawdown of the water level inside the well is measured during each stage. In general not less than 3 but preferably 4 stages are required, the largest capacity being at least 20% more than the anticipated permanent capacity. Water levels in the well are measured, first at brief intervals and later on with longer periods in between, until the water level remains virtually constant. The capacity is then increased to the next value and the procedure repeated (Fig.3A). After the last step has been completed, pumping is stopped and the recovery of the water level in the well is measured. From the re-

sults of this step-wise pumping, the capacity-drawdown relationship can be determined (Fig.3B).

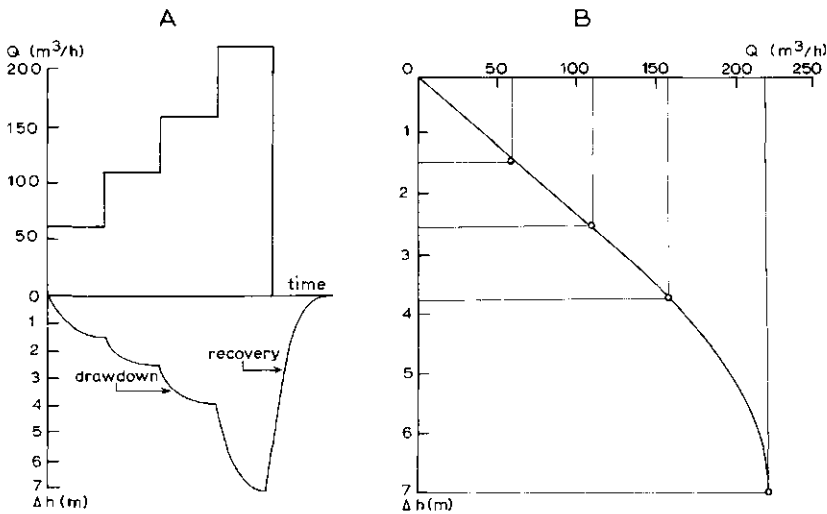


Fig.3. Well testing. A. Step-wise pumping of a well. B. Discharge-drawdown relationship

The pumping rate selected is that which produces measurable drawdowns within a radius of, say, 500 m from the well. The maximum permissible rate is the highest value for which the straight-line relationship between discharge and drawdown in the well applies. As can be seen from Fig.3B, exceeding the maximum pumping rate by even a small amount results in a large increase in the drawdown. During the actual test the well should be operated at its maximum capacity to reduce as much as possible the effect of water level changes not due to pumping. A deviation of 2 cm on a drawdown of 10 cm means an error of 20% but this value drops to only 5% if the pumping rate is quadrupled.

25.1.5 PIEZOMETERS AND OBSERVATION WELLS

Once the site of the discharging well has been selected, the number and depth of the piezometers or observation wells, and their distances from the discharging well must be decided.

A piezometer (Chap.21, Vol.III) is an open-ended unperforated pipe placed in the ground and extending from the surface to the depth at which the hydraulic head is to be measured (Fig.4). The term applies equally well to pipes that have a short screen attached to their lower end. An observation well is an open bore hole, or a bore hole provided with a pipe which is perforated or slotted over a relatively great length. The water level in such a well therefore represents the average hydraulic head over the net length of the bore hole (depth of inflow) or over the length of the perforated or slotted portion of the pipe. Open bore holes with or without pipes are used to observe changes in the phreatic level at shallow depth.

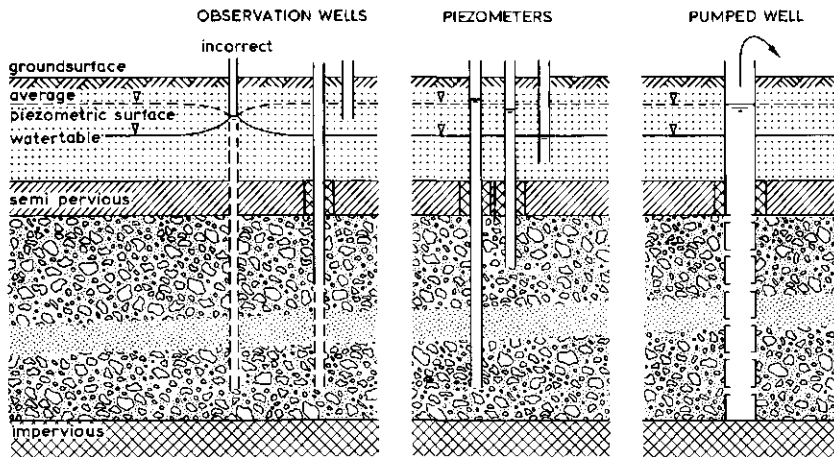


Fig.4. Position of screen in observation wells, piezometers, and discharge well

Number and depth

Most methods of analysis require data from at least two piezometers whose lower ends are located in the aquifer. It is recommended, however, that three or more be installed. To these should be added, if applicable, some or all of the following:

- shallow observation wells in the covering layer of semi-confined aquifers, to check the changes in the phreatic surface;

Aquifer characteristics

- piezometers above and below fine-grained intercalations in the aquifer, to find out whether such intercalations are to be considered resistance layers or not;
- piezometers in water-bearing layers below the base of the aquifer, to verify the assumption that this base is impervious;
- a piezometer beyond the radius of influence of the discharge well, to obtain information on the changes in head not induced by the pumping;
- a piezometer in the gravel pack of the discharge well, to determine whether the well screen causes significant entrance head losses.

Distances

No fixed rule can be given for the distances at which the observation wells or piezometers should be installed. The best guide is the experience gained during pumping tests in comparable aquifers. Nevertheless, the following points should be considered.

The type of aquifer

In unconfined (phreatic) aquifers the propagation of the head loss is rather slow because the water released from storage is due mostly to a dewatering of the zone above the falling water table. The piezometers should therefore not be too far away from the discharging well.

Semi-confined aquifers react faster (depending on the hydraulic resistance of the semi-pervious layer) so the distances between piezometers and the pumped well can be greater.

The value of the hydraulic conductivity K and the storage coefficient S

If K is large and S is small, the cone of depression created by pumping is wide and flat; if K is small and S is large, the cone is narrow and steep. Thus, in the first case greater distances are permitted than in the second.

The discharge rate

Pumping at the maximum discharge rate will cause a greater drawdown than pumping at a low discharge rate. In the first case, therefore, the piezometers can be placed farther away than in the second.

The position of the well screen

If the discharging well is fully penetrating, water will flow to the well horizontally and reliable drawdown observations can be made, even at a short distance from the well. If, on the other hand, the well is partially penetrating, the nearest piezometer should be installed at a distance from the pumped well which is at least as great as the aquifer is thick. Closer to the well, vertical flow components influence the drawdown and rather complicated correction methods have to be applied (KRUSEMAN and DE RIDDER, 1970; HUISMAN, 1972).

Although much depends on local conditions, the following distances for a series of six piezometers in a semi-confined aquifer may serve as a guide (see also Fig.5).

Piezometer

- 1 < 1 m from the well (i.e. in gravel pack)
- 2 1 - 3 m from the well
- 3 3 - 10 m from the well
- 4 10 - 30 m from the well
- 5 30 - 100 m from the well
- 6 > 300 m from the well (beyond radius of influence)

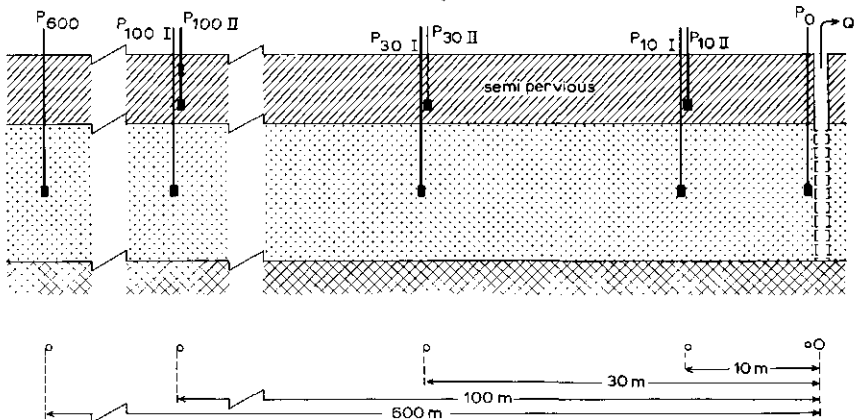


Fig.5. Example of piezometer arrangement. First index refers to distance in m from pumped well, second index indicates layer in which screen is placed

When a large number of piezometers are to be placed, they should be arranged in two lines perpendicular to another, their point of intersection coaxial with the pumped well. In this way the occurrence of directional anisotropy in the aquifer, if any, can be detected. In isotropic aquifers lines of equal drawdown are circles whereas in aquifers with directional anisotropy they are ellipses (HANTUSH and THOMAS, 1966).

25.2 PERFORMING A PUMPING TEST

The measurements to be taken during a pumping test can be divided into two groups:

- measurements of the water levels in the wells and piezometers,
- measurements of the pumping rate.

Since the natural position of the water table will be affected by changes in barometric pressure and in water levels of nearby surface waters, by natural groundwater discharge and by precipitation, these should be measured if the pumping test is expected to last a long period. Anomalies observed in the drawdown values can then be explained and corrected.

25.2.1 WATER LEVEL MEASUREMENTS

Measurements prior to and after the pumping test

The natural changes in hydraulic head occurring in the aquifer, i.e. those changes not induced by pumping, should be known before a pumping test is started. Hence, for some days prior to the test, the water levels in the observation wells and/or piezometers should be measured twice a day if a unidirectional change is expected, or at hourly intervals if a sinusoidal fluctuation pattern is expected. The latter is a common feature in coastal aquifers and in valleys with river levels subject to tidal movements.

A hydrograph should be drawn for each observation well, from which the trend and rate of change in hydraulic head can be read. If the hydrograph indicates that the trend will not change during the test, pumping can start. When the test is completed and the water level has been fully recovered, water level readings should be made in some observation wells once or twice a day for two or three days. These data complete the well hydrograph and the rate of the natural change in hydraulic head during the test can then be determined. This information is used to correct the observed drawdowns into drawdowns induced by pumping alone.

Frequency of observations

The most important part of a pumping test is measuring the depth to water level in the observation wells, piezometers, and in the pumped well. These measurements should be taken frequently, and should be as accurate as possible. Since water levels are dropping fast during the first one or two hours of the test, they should be recorded at close intervals throughout that period. The time intervals may be gradually lengthened as the test continues because in the analysis the

factor time generally enters in a logarithmic form. Table 1 shows a practical range of intervals between measurements in piezometers close to the pumped well where the reaction is fastest. This range may have to be adapted to suit local conditions, available manpower, etc.

Table 1. Range of time intervals between measurements in piezometers close to the pumped well.

Time since pumping started	Time intervals
0 - 2 minutes	approx. 10 seconds
2 - 5 minutes	30 seconds
5 - 15 minutes	1 minute
15 - 50 minutes	5 minutes
50 - 100 minutes	10 minutes
100 min. - 5 hours	30 minutes
5 hours - 48 hours	60 minutes
48 hours - 6 days	3 times per day
6 days - shut down of the pump	once a day

For piezometers at greater distances from the well and for those in semi-pervious layers above or below the aquifer, measurements need not be taken at such brief intervals during the first minutes of the test. When there are two or more piezometers in the immediate vicinity of the well, the intervals between measurements in the well itself may also be extended, e.g. to one hour.

Measuring equipment

The depth to the water level in piezometers can be accurately measured with such simple devices as:

- a steel tape, float, and standard with pointer
- an electrical or mechanical sounder
- a wetted tape.

These methods (Chap.21, Vol.III) allow an accuracy of one or two millimeters and are convenient for measuring the rapidly changing water levels that occur close to the pumped well.

Aquifer characteristics

Water level recorders greatly reduce the amount of work required and allow it to be better organized. When used in piezometers close to the well, however, the curves obtained will be very steep during the first minutes of the test and can be interpreted only if the time intervals have been marked by hand on the graph. Detailed information on automatic recorders, mechanical and electrical sounders and other equipment is given in such handbooks as JOHNSON (1966), DAVIS and DE WIEST (1966).

All water level measurements should preferably be noted on pre-printed forms, an example of which is shown in Fig.6. During the pumping test, time-drawdown curves for each piezometer are drawn on semi-logarithmic paper (the time in minutes on the logarithmic scale, and the drawdown in centimetres or millimetres on the linear scale). These graphs may be helpful in seeing whether the test is running well and in deciding on the time the pump is to be shut down.

Recovery measurements

When the pump is shut down, the water levels in the pumped well and the piezometers will rise rapidly, but as time goes on the rate of rise decreases. Measuring the rate of rise is called the recovery test. An analysis of the data of such a test can be used to check the calculations based on the drawdown data.

If the yield of the pumped well was not constant throughout the pumping period, recovery test data are more reliable than the drawdown data collected during pumping. The recovery measurements should follow the same schedule as adhered to during the pumping period.

25.2.2 DISCHARGE RATE MEASUREMENTS

The discharge rate should be measured at least once every hour and adjusted, if necessary, to keep it constant. Adjustments can be made by means of a valve in the discharge pipe or by a change in the speed of the pump. The valve, however, permits a more precise control.

Accurate measurements of the discharge rate can be made with a commercial water meter of appropriate capacity, if the pipe is running full. To ensure this, the discharge pipe may be fitted with a U-bend in which the water meter is housed. This will, however, induce energy losses. If the pumped water is being conveyed through a channel or small ditch, the discharge rate can be measured by means of a flume or a weir. A very simple and fairly accurate method is to measure the time required to fill a container of known volume, e.g. an oil drum. This method, however, can only be applied if the discharge rates are small.

PUMPING TEST:.....

OBSERVATIONS DURING PUMPING / RECOVERY

PIEZOMETER:..... DEPTH:..... DISTANCE:.....

PUMPING TEST BY:..... DIRECTED BY:.....

PROJECT:..... LOCATION:.....

START:..... STOP:.....

INITIAL WATER LEVEL:..... FINAL WATER LEVEL:.....

REFERENCE LEVEL:..... ± M.S.L.¹

REMARKS:.....

TIME	WATER LEVEL	DRAW DOWN	TIME	WATER LEVEL	DRAW DOWN	DISCHARGE RATE		
						TIME	FLOW METER	DISCHARGE RATE

¹ MEAN SEA LEVEL

Fig.6. Example of a pumping test observation sheet

If none of these methods can be applied, a rough estimate can be obtained by measuring the dimensions of a stream flowing from either a vertical or horizontal discharge pipe (JOHNSON, 1966). With a vertical pipe the discharge rate can be calculated from the height to which the water rises above the top of the pipe and the diameter of the pipe. The discharge rates for various pipe diameters and various crest heights are given in Fig.7. With a horizontal pipe, flowing full

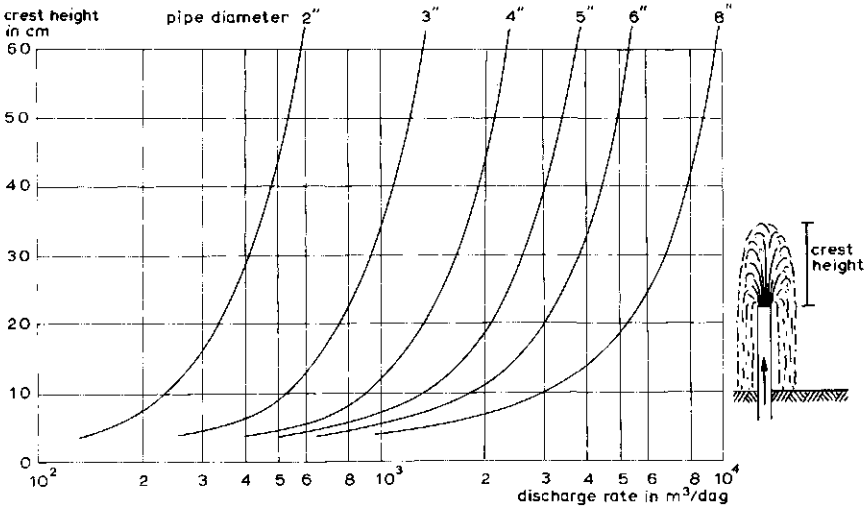


Fig.7. Relation between discharge rate, jet flow crest height, and pipe diameter (after JOHNSON, 1966).

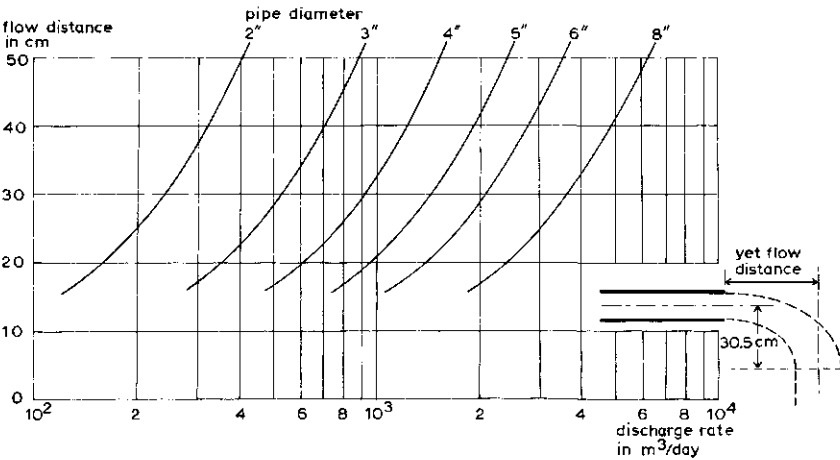


Fig.8. Relation between discharge rate, jet flow distance, and pipe diameter for a fall of 30.5 cm (=12") (after JOHNSON, 1966)

and allowing a free fall from the discharge opening, the horizontal and vertical distances from the end of the pipe to a point in the falling stream provide the basis for calculating the discharge rate. Figure 8 shows the pumping rate for various pipe diameters and horizontal distances at a given fall.

25.2.3 DURATION OF THE PUMPING TEST

The question of how many hours the well should be pumped is difficult to answer because the period of pumping depends on the type of aquifer being tested. As already observed, the cone of depression develops rapidly at the beginning of the test but as pumping continues, it expands and deepens at a decreasing rate because a larger volume of stored water becomes available with each additional meter of horizontal expansion. This results in a decrease in drawdown per unit of time, which may lead to the premature conclusion that the cone has reached a stabilized position, i.e. that steady-state flow conditions have been reached. In some aquifers an equilibrium drawdown may be reached a few hours after the start of pumping. In others it may be reached after a few days or weeks or it may never be reached. In semi-confined aquifers a steady-state flow situation generally occurs after 15 to 20 hours of pumping. Since in an unconfined aquifer the cone of depression expands more slowly, it is common practice to pump such an aquifer for about three days.

Although methods of analyzing unsteady-state data are available, it is recommended that pumping be continued till a steady state is reached. This is especially warranted when accurate information on the aquifer characteristics is required, for instance when pumping plants for domestic or irrigation water supplies are to be constructed or when drainage wells are to be installed or other expensive works are to be carried out. Moreover, a longer period of pumping may reveal the presence of hydraulic boundaries, previously unknown. The cost of running the pump a little longer is low compared with the total cost of the test.

25.3 DATA PROCESSING

After the pumping test has been completed and all basic information on well discharge, drawdown in the various piezometers and pumped well, trend of the natural changes in hydraulic head, etc., has been collected, the data must then be processed. This comprises:

Aquifer characteristics

- compiling the data in the form of graphs;
- correcting the drawdown data for changes of the hydraulic head in the aquifer not induced by pumping, if applicable;
- determining the type of aquifer that has been pumped.

The field information that consists of time data often given in different units, (seconds during the first minutes, minutes during the following hours and actual time later on) and water level observations relative to a reference level, must be converted into data of a single set of units (for instance time in minutes and drawdown in metres). These data are then noted on a blank set of observation sheets (Fig.6), together with all other relevant data.

The drawdowns observed in each piezometer during pumping are plotted versus the corresponding time data on double logarithmic paper and/or on single logarithmic paper. The observations made prior to pumping and after complete recovery are plotted on paper with linear scales to check whether any changes in head have taken place. In the graph of the "long distance" piezometer the observations made during the pumping and recovery periods should be included as well. From these graphs the correction term, i.e. the rate of rise or fall per minute, can be calculated. Next, the observed pumping and recovery data are corrected and the time-drawdown graphs are plotted again, this time with the corrected data.

The type of aquifer is determined from: the bore-hole log, the time-drawdown curves of the piezometers in the aquifer, and the reaction to pumping observed in the auxiliary piezometers in the covering layer and in those below the base layer.

If the base layer is impervious (no reaction to pumping in the piezometers below this layer) and there is a definite covering layer, the drawdowns observed in the piezometers in the covering layer will help to determine the type of aquifer.

If the drawdown in the shallow observation wells is approximately equal to the drawdown of the deep piezometers, the aquifer is considered to be unconfined. If there is an appreciable difference between the drawdowns, the aquifer is considered to be semi-confined.

25.4 CALCULATION OF THE HYDRAULIC CHARACTERISTICS

In discussing the analysis of pumping tests, we shall restrict our remarks to horizontal, seemingly infinite, unconfined or semi-confined aquifers. A comprehensive review of methods of analyzing tests in other types of aquifers is given by KRUSEMAN and DE RIDDER (1970).

25.4.1 UNCONFINED AQUIFERS, STEADY FLOW

According to Chap.12, Vol.II, when the flow towards the well was in a steady state at shut down of the pump, the logarithmic function for this flow around the well in an unconfined aquifer is written as

$$\Delta h_1 - \Delta h_2 = \frac{Q}{2\pi KD} 2.30 \log \frac{r_2}{r_1} \quad (1)$$

where (see Fig.9)

- $\Delta h_1, r_1$ = steady-state drawdown at distance r_1 (m),
- $\Delta h_2, r_2$ = steady-state drawdown at distance r_2 (m),
- Q = constant well discharge ($\text{m}^3 \text{ day}^{-1}$),
- KD = transmissivity of the aquifer ($\text{m}^2 \text{ day}^{-1}$).

Substitution of the values $\Delta h_1, \Delta h_2, r_1, r_2,$ and Q into Eq.1 yields the value for KD .

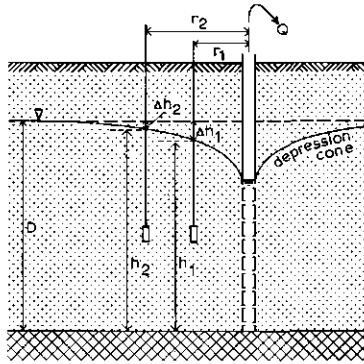


Fig.9. A pumped well and two piezometers in an unconfined aquifer

25.4.2 UNCONFINED AQUIFERS, UNSTEADY FLOW

When the flow towards the well was still unsteady at shut down of the pump, the analysis of the data is less simple.

Two graphical methods can be followed

- Theis's double-logarithmic method
- Jacob's single-logarithmic method

Theis's double logarithmic method

This method, first described by COOPER and JACOB (1946), is based on the fact that the unsteady-state drawdown Δh , which is a function of r and t , is proportional to another function in r and t , i.e. the well function $W(u)$. According to Chap.12, Vol.II,

$$\Delta h = \frac{Q}{4\pi KD} W(u) \quad (2)$$

where

$$u = \frac{r^2 S}{4KDt} \quad (3)$$

S = storage coefficient (dimensionless)

t = time in days

The procedure to be followed in the analysis is as follows:

- Construct, with the aid of Table 2, a 'master-chart' by plotting the values of u on the horizontal axis of a sheet of double logarithmic paper against the corresponding values of $W(u)$ on the vertical axis.
- Plot on another sheet of double-logarithmic paper of the same scale, but now preferably transparent, each observed value of Δh on the vertical axis against its corresponding value of r^2/t . If there are n piezometers and m observations in each piezometer, then $m \times n$ observations should be used.
- Place the transparent paper with the plotted observations on the master-chart and shift till the observed data curve matches the curve on the master-chart.
- Select arbitrarily a match point on the overlapping sheets and determine for this point the coordinates $W(u)$, u , Δh , and r^2/t . Note that the match point is not necessarily located on the curves.
- Substitute the values of $W(u)$, Δh , and Q into Eq.2 and solve for KD .
- Substitute the values of u , r^2/t , and KD into Eq.3 and solve for S :

Example 1

During a pumping test, 2500 m³/day was pumped from a well in an unconfined aquifer. The following drawdown and time observations were made in two piezometers, 10 m and 100 m from the well.

Piezometer at 10 m

Δh	time since pumping started		r^2/t	
0.25 m	57 min. 30 sec.	= 0.04 day	100/0.04	= 2500 m ² /day
0.36 m	3 hours 50 min.	= 0.16 day	100/0.16	= 625 m ² /day
0.50 m	24 hours	= 1 day	100/1	= 100 m ² /day

Piezometer at 100 m

0.06 m	6 hours	= 0.25 day	10.000/0.25	= 40000 m ² /day
0.10 m	10 hours 40 min.	= 0.444 day	10.000/0.444	= 22500 m ² /day
0.15 m	24 hours	= 1 day	10.000/1	= 10000 m ² /day

Table 2. Values of Theis's well function

u	W(u)	u	W(u)
0.0001	8.63	0.10	1.82
0.0002	7.94	0.20	1.22
0.0004	7.25	0.40	0.702
0.0006	6.84	0.60	0.454
0.0008	6.57	0.80	0.311
0.001	6.33	1.0	0.219
0.002	5.64	1.2	0.158
0.004	4.94	1.4	0.116
0.006	4.54	1.6	0.0863
0.008	4.26	1.8	0.0647
0.01	4.04	2.0	0.0489
0.02	3.35	2.5	0.0249
0.04	2.68	3.0	0.0131
0.06	2.29	3.5	0.00697
0.08	2.03	4.0	0.00378

The observed-data curve and the master-chart curve in matched position are presented in Fig.10. Point A has been selected as match point, its master-chart coördinates being $W(u) = 1$ and $u = 10^{-2}$, and its data-sheet coördinates $\Delta h = 0.076$ and $r^2/t = 1200$.

Aquifer characteristics

Substitution of the appropriate values in Eqs.2 and 3 gives

$$KD = \frac{Q}{4\pi\Delta h} W(u) = \frac{2500}{4 \times 3.14 \times 0.076} \times 1 \approx 2600 \text{ m}^2/\text{day}$$

$$S = \frac{4KD u}{r^2/c} = \frac{4 \times 2600 \times 10^{-2}}{1200} \approx 0.09$$

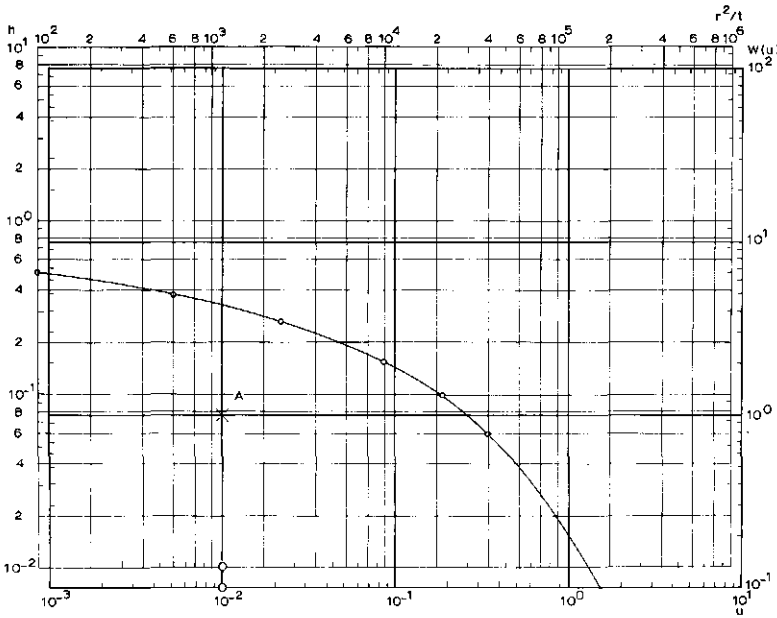


Fig.10. Theis's double-logarithmic method

Jacob's single-logarithmic method

This method (COOPER and JACOB, 1946) is based on Theis's method (Chap.14, Vol.II)

$$\Delta h = \frac{Q}{4\pi KD} 2.30 \log \frac{2.25KDt}{r^2 S} \tag{4}$$

Since $\frac{2.30Q}{4\pi KD} \neq 0$ it follows that

$$\log \frac{2.25KDt_0}{r^2 S} = 0$$

or

$$\frac{2.25KDt_0}{r^2S} = 1 \quad (5)$$

where $t_0 = t$ for $\Delta h = 0$.

The slope of the straight line is

$$\frac{\Delta h_2 - \Delta h_1}{\log t_2 - \log t_1} = \frac{2.30Q}{4\pi KD}$$

It is practical to solve this equation for $\Delta h_2 - \Delta h_1$ by taking $\log t_2 - \log t_1 = 1$ which is the same as $t_2 = 10 t_1$. The equation, then, reduces to

$$\Delta(\Delta h) = \frac{2.30Q}{4\pi KD} \quad (6)$$

where $\Delta(\Delta h)$ = the drawdown difference ($\Delta h_2 - \Delta h_1$) per log cycle of time. Therefore, proceed further as follows:

- Select a length on the t -axis which equals one log cycle, e.g. between $t = 2$ and $t = 20$ or between $t = 0.3$ and $t = 3$, and read $\Delta(\Delta h)$.
- Substitute the values of Q and $\Delta(\Delta h)$ in Eq.6 and solve for KD .
- Substitute the values of KD , t_0 , and r in Eq.5 and solve for S .

This procedure should be repeated for all piezometers. The calculated values of KD and S should show a close agreement.

Example 2

During a pumping test in an unconfined aquifer, a piezometer 5 m from the pumped well showed the following drawdown development at a pumping rate of $120 \text{ m}^3/\text{hour} = 2880 \text{ m}^3/\text{day}$.

$t = 2$	4	8	18	30	60 min
$\Delta h = 0.27$	0.37	0.47	0.58	0.66	0.73 metre

A plot of Δh versus t on single logarithmic paper yields (Fig.11)

$$t_0 = 0.32 \text{ min} = 2.22 \times 10^{-4} \text{ day}$$

$$\Delta(\Delta h) = 0.33 \text{ m}$$

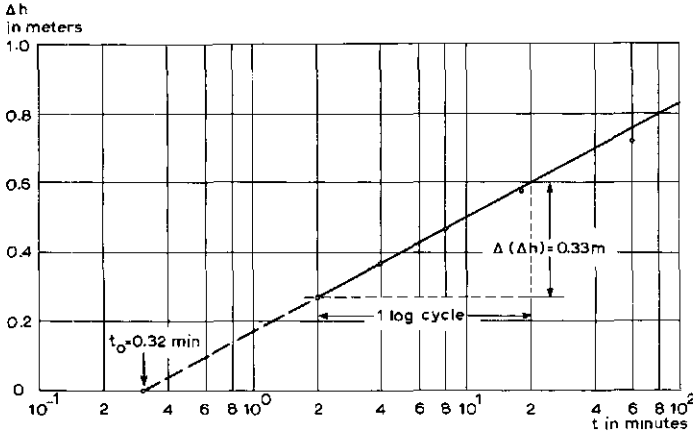


Fig.11. Jacob's single logarithmic method

Substitution of the appropriate values into Eqs.6 and 5 yields

$$KD = \frac{2.30Q}{4\pi\Delta(\Delta h)} = \frac{2.30 \times 2880}{4 \times 3.14 \times 0.33} \approx 1600 \text{ m}^2/\text{day}$$

$$S = \frac{2.25KDt_0}{r^2} = \frac{2.25 \times 1600 \times 2.22 \times 10^{-4}}{5 \times 5} \approx 0.03$$

Analysis of recovery data

As was shown in Chap.12, Vol.II, the drawdown development after the pump is shut down can be found by adding to the drawdown effected by the discharge rate Q, the (negative) drawdown that would be caused if the pumped well were recharged at the same rate as it was pumped (Fig.12). Hence, according to Chap.12

$$\Delta h' = \frac{Q}{4\pi KD} \ln \frac{4KDt}{r^2 S} + \frac{-Q}{4\pi KD} \ln \frac{4KDt'}{r^2 S}$$

or, since

$$\ln \frac{4KDt}{r^2 S} - \ln \frac{4KDt'}{r^2 S} \approx \ln(t/t')$$

$$\Delta h' = \frac{Q}{4\pi KD} \ln \frac{t}{t'} = \frac{Q}{4\pi KD} 2.3 \log(t/t')$$

where

$\Delta h'$ = the residual drawdown in m, i.e. the difference between the initial water level prior to pumping and the actual water level at time t' since pumping stopped

t = time since pumping started, in days

t' = time since pumping stopped, in days

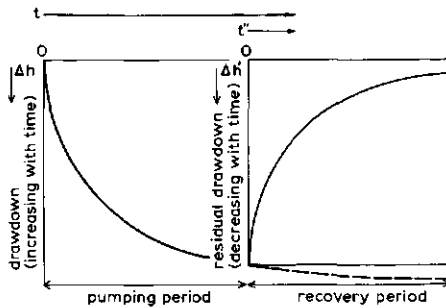


Fig.12. Schematic drawdown development during pumping and recovery period. The broken line indicates the continuing influence of imaginary pumping when imaginary recharge simulates shutdown of pump

Procedure

- Plot for one of the piezometers or for the pumped well, $\Delta h'$ versus (t/t') on single logarithmic paper (t/t' on the logarithmic axis) and draw a straight line through the plotted points. Where this straight line intersects the t/t' -axis, i.e. the point where $\Delta h' = 0$, the point $t/t' = 1$. Following the same reasoning as for Jacob's method we get

$$\Delta(\Delta h') = \frac{2.30Q}{4\pi KD} \tag{7}$$

where $\Delta(\Delta h')$ is the residual drawdown difference per log cycle of t/t' .

- Substitute the values of $\Delta(\Delta h')$ and Q into Eq.7 and solve for KD .

Example 3

The pump of a well in an unconfined aquifer is shut down after 24 hours (= 1440 min) of pumping at a rate $Q = 83.5 \text{ m}^3/\text{hour}$ (= $2000 \text{ m}^3/\text{day}$).

The observed decrease of the residual drawdown is as follows

t' (in min)	2	5	15	30	75	150	360	720	1440
$\Delta h'$ (in min)	0.84	0.78	0.68	0.58	0.46	0.35	0.24	0.16	0.11
t/t'	721 ¹⁾	290	97	49	20	10.6	5	3	2

¹⁾ $t = 1440 + t'$

A plot of $\Delta h'$ versus t/t' on single logarithmic paper (Fig.13) yields

$$\Delta(\Delta h') = 0.34$$

Substituting the appropriate values into Eq.7 and solving for KD gives

$$KD = \frac{2.30Q}{4\pi\Delta(\Delta h')} = \frac{2.30 \times 2000}{4 \times 3.14 \times 0.34} \approx 1100 \text{ m}^2/\text{day}$$

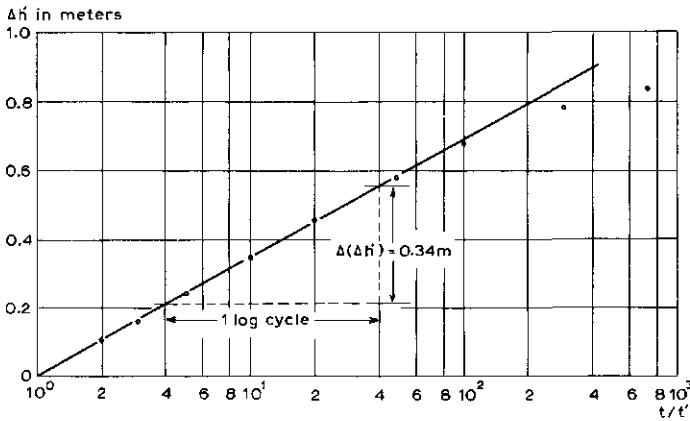


Fig.13. Analysis of recovery data

25.4.3 SEMI-CONFINED AQUIFERS, STEADY FLOW

Two graphical methods will be discussed

- DE GLEE's double-logarithmic method
- HANTUSH-JACOB's single-logarithmic method.

DE GLEE's double-logarithmic method

This method, originally described by DE GLEE (1930), is based on the following relation (Chap.12, Vol.II)

$$\Delta h = \frac{Q}{2\pi KD} K_0(r/\sqrt{KDc}) \tag{8}$$

where

- $K_0(x)$ = modified Bessel function of the second kind and zero order (Hankel function)
 Δh = steady-state drawdown at distance r from the pumped well (m)
 c = D'/K' = hydraulic resistance to vertical flow in the semi-pervious layer (days)
 KD = transmissivity of the pervious layer ($m^2 \text{ day}^{-1}$)

Procedure

- Construct, with the aid of Table 3, a master-chart by plotting values of $K_0(x)$ against x on a sheet of double logarithmic paper.
- Plot on another sheet of double logarithmic paper, of the same scale but preferably transparent, the observed steady-state drawdown values against the corresponding values of r . Each piezometer has only one steady-state drawdown value; hence, if there are n piezometers then there are also n observations to be used.
- Match both curves and select arbitrarily a point A on the overlapping portion of the sheets.
- Read the coördinates $K_0(x)$, x , Δh , and r of Point A.
- Substitute the values of Δh , $K_0(x)$, and Q into Eq.8 and solve for KD .

Note that

$$x = r/\sqrt{KDc} \quad (9)$$

- Substitute therefore the values of x , r , and KD into Eq.9 and solve for c .

Table 3. Values of the modified Bessel function of the second kind and zero order

x	$K_0(x)$	x	$K_0(x)$
0.01	4.72	1.0	0.421
0.02	4.03	1.2	0.318
0.04	3.34	1.4	0.244
0.06	2.93	1.6	0.188
0.08	2.65	1.8	0.146
0.1	2.43	2.0	0.114
0.2	1.75	2.5	0.0623
0.4	1.11	3.0	0.0347
0.6	0.777	3.5	0.0196
0.8	0.565	4.0	0.0112

Example 4

From a well in a semi-confined aquifer water has been pumped at a rate of $100 \text{ m}^3/\text{hour} = 2400 \text{ m}^3/\text{day}$. The following steady-state drawdown observations have been made

distance	25	50	150	250 m
drawdown	0.31	0.22	0.09	0.05 m

The observed data curve and the master-chart curve in matched positions are presented in Fig.14.

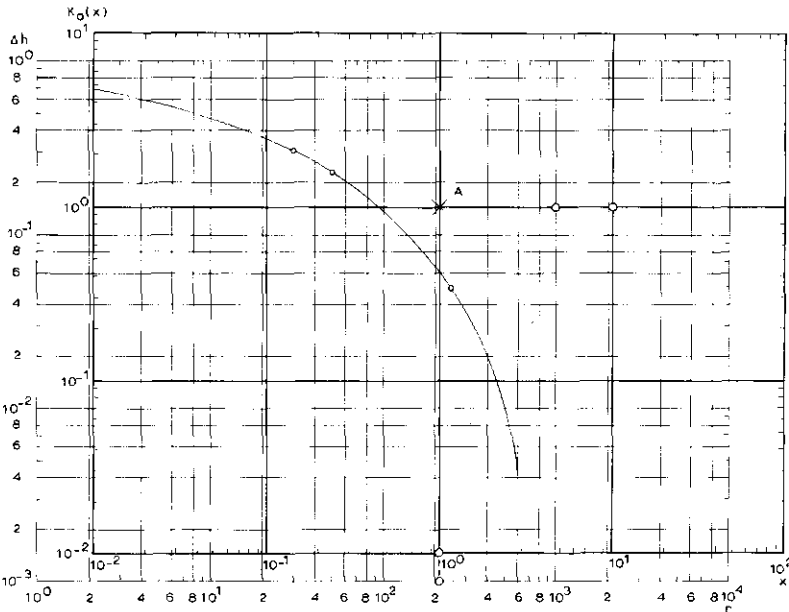


Fig.14. DE GLEE's double-logarithmic method

Point A has been selected as match point, its master-chart coordinates being $K_0(x) = 1$ and $x = 10^0 = 1$ and its observed data sheet coordinates $\Delta h = 0.145$ and $r = 210$.

Substitution of the appropriate values in Eqs.8 and 9 gives

$$KD = \frac{Q}{2\pi\Delta h} K_o(x) = \frac{2400}{2 \times 3.14 \times 0.145} \times 1 \approx 2600 \text{ m}^2/\text{day}$$

$$c = \frac{r^2}{KDx^2} = \frac{210^2}{2600 \times 1^2} \approx 17 \text{ days}$$

HANTUSH-JACOB' single logarithmic method

HANTUSH (1956) showed that for $r/\sqrt{KDC} < 0.05$, Eq. 8 may be approximated by

$$\Delta h = \frac{Q}{2\pi KD} \ln 1.12 \frac{\sqrt{KDC}}{r}$$

or with the logarithm on the basis 10

$$\frac{Q}{2\pi KD} 2.30 \log 1.12 \frac{\sqrt{KDC}}{r} \tag{10}$$

where the symbols are as defined above.

Procedure

- Plot the steady-state drawdown values Δh versus r on single logarithmic paper (r on the logarithmic axis) and draw a straight line through the plotted points; where this line intersects the r -axis (i.e. the line where $\Delta h = 0$), $r = r_o$.

Following the same reasoning as for the Jacob method one gets

$$\frac{1.12 \sqrt{KDC}}{r_o} = 1 \tag{11}$$

and

$$\Delta(\Delta h) = \frac{2.30Q}{2\pi KD} \tag{12}$$

where $\Delta(\Delta h)$ is the drawdown difference per log cycle of r .

- Substitute the values of Q and $\Delta(\Delta h)$ in Eq.12 and solve for KD .

- Substitute the values of KD and r_o in Eq.11 and solve for c .

Example 5

Plotting the data used in Example 4 on single logarithmic paper results in (Fig.15)

$$r_o = 320 \text{ m}$$

$$\Delta(\Delta h) = 0.285 \text{ m}$$

Substitution of the appropriate values into Eqs.11 and 12 yields

$$KD = \frac{2.300}{2\pi\Delta(\Delta h)} = \frac{2.30 \times 2400}{2 \times 3.14 \times 0.285} = 3100 \text{ m}^2/\text{day}$$

$$c = \frac{1}{KD} \left(\frac{r_o}{1.12}\right)^2 = \frac{1}{3100} \times \left(\frac{320}{1.12}\right)^2 = 26 \text{ days}$$

The results with the De Glee method differ from those obtained with the Hantush-Jacob method, which is illustrative of the inaccuracies inherent to these graphical methods.

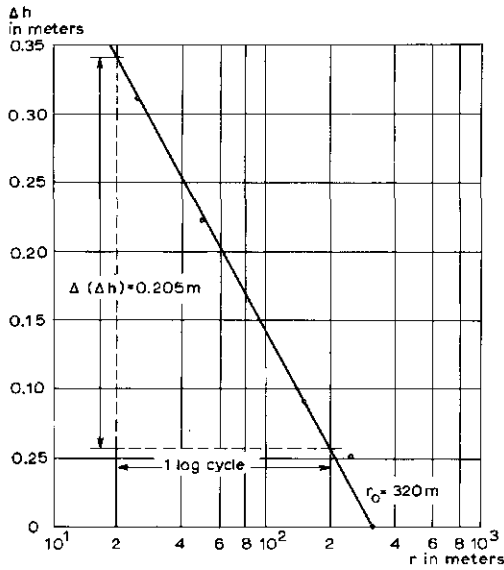


Fig.15. Hantush-Jacob's single-logarithmic method

25.5 FINAL REMARKS

Before any of the methods of analysis is applied, attention should be given to the assumptions underlying that particular method.

The natural conditions nearly always deviate somewhat from the assumed ones.

These deviations should be taken into account when the final evaluation of the calculations is being made. Several methods of analysis, if applicable, should be used to calculate the hydraulic characteristics. The results should be compared and weighed by one's professional judgement when the mean values are being calculated.

Sometimes the observed data show unexpected anomalies. Some of them are mentioned below:

- A great difference between the water levels in the pumped well and at the well face. This is a common phenomenon due to faulty construction of the well, or its insufficient development. The result is a high entrance resistance into the well, which causes the anomalous head loss. This is one of the reasons why piezometer data, rather than data from the pumped well, are preferred for calculation purposes.
- A smaller drawdown in a piezometer close to the discharging well than in one farther away. This may be due to inhomogeneities of the aquifer which should, however, have been revealed from the study of the well logs. Other explanations may be found in the influence of a second pumped well in the vicinity, the lowering of the level of a nearby canal, etc.

Variations in the hydraulic resistance of a semi-pervious layer may also have an effect on piezometer readings.

Variations in the discharge rate will have a greater effect on piezometers near the pumped well than on those farther away. These influences can easily be traced by an examination of the measured discharge rate. Faulty piezometers and water-level indicators and inattentive observers are other sources of deviations from the expected course of events. Testing of piezometers and equipment during development pumping and the use of well-trained personnel will largely overcome these problems.

A report should be written at the conclusion of the pumping test, and should contain the following items:

- a map of the pumping test site and surrounding area, including locations of main well, piezometers, and open water courses, if any;
- a lithological cross-section of the test site, based on data from the borings;
- sheets of the observed field data;
- a description and/or graphs, illustrating the corrections applied to the ob-

Aquifer characteristics

served data, if applicable;

- a conclusion regarding the type of aquifer and the considerations that led to the selection of the analytical methods used;
- graphs of the time-drawdown curves in superposition on the type curves, if applicable;
- the calculations in an abbreviated form;
- the values of the hydraulic characteristics of the aquifer and confining layers, as well as a discussion as to their accuracy;
- recommendations for further investigations, if applicable;
- a summary of the results obtained.

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SURVEYS AND INVESTIGATIONS

26. DERIVING SOIL HYDROLOGICAL CONSTANTS
FROM FIELD DRAINAGE TESTS

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PURPOSE AND SCOPE

The layout of experimental fields, the network of observations, and the processing of measurements with a view to obtaining information on the soil hydraulic conductivity, the transmissivity, the effective porosity, and the groundwater reservoir-coefficient.

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26.1 OBJECTIVES OF FIELD EXPERIMENTS

The measurement of such hydrologic soil characteristics as hydraulic conductivity of the upper layer, transmissivity of a phreatic aquifer, and the effective porosity has been discussed in Chap.24, Vol.III. It will have been observed that the volume of soil involved in the measurement is usually small. As an example, when hydraulic conductivity is measured by the auger hole method, the soil volume that contributes to the measurement seldom exceeds 0.5 m^3 . The sample sizes for laboratory methods are even smaller. Due to this limited volume of soil investigated per observation a large number of measurements is needed to obtain a statistically reliable average that can be used for the design of the drainage system, but often a much smaller number is taken in practice. As a consequence, it is often desirable - and sometimes even indispensable - to make use of experimental fields to verify the initial assumptions concerning the hydrological soil properties on which the design of the drainage system has to be based. In areas where soil conditions are rather heterogeneous, experimental fields may be the only or most efficient way to obtain the required information on drainage design factors. Such fields, moreover, may serve to collect information on individual water balance factors, the desirable watertable depth, the efficiency of the irrigation system, the leaching efficiency, etc. They may also be used to test drainage materials, such as pipes and filter materials, or new techniques of drainpipe installation. Before constructing the field one should carefully define the problem to be investigated. The layout of the plots and the network of observations should be such that adequate information can be obtained and that as many subjects of interest as possible are covered.

26.2 SCOPE OF PRESENT DISCUSSION

The present discussion will be limited to the layout of trial fields, the measurements and the processing of field data in view of obtaining information on the hydraulic conductivity and transmissivity of the soil, the effective porosity in the zone of fluctuating water tables and the groundwater reservoir-coefficient.

The layout and observation systems suggested in the following sections will in principle be suitable for the other tests above.

26.3 SELECTION OF TRIAL SITE

The site of a experimental field should be chosen in such a way that the groundwater table is only influenced by precipitation, irrigation and evaporation. The results obtained from experimental fields should be applicable to an area of fair

size. Therefore the hydrologic, soil, topographic conditions etc., should be representative of those prevailing in the area under investigation.

26.4 SIZE OF EXPERIMENTAL UNIT

The various trial plots together make up the experimental trial unit. Its desirable size depends largely on the nature of the problem under investigation and related conditions, but is likely to be less than 50 ha. Often an area of 10-30 ha will be sufficient in first instance. Small units offer considerable advantages from the viewpoint of organization of observations and processing data.

It is better to have reliable observations from a small area than infrequent or poor quality observations from a larger one.

26.5 DIMENSIONS OF INDIVIDUAL TRIAL PLOTS

The dimensions are governed by

- the drain spacing to be tested; spacings which are narrower and wider than those calculated or estimated should be included as well and the intervals should be chosen in distinct steps. If for example calculations indicate a spacing of 50 m, include spacings of 25 and 100 m,

- a length-width ratio of at least 5 and preferably 7-10 or more. Thus, when the width (i.e. the spacing) is 50 m, the length should be at least 250 m and preferably 350-500 m or more.

Note that due to considerable border effects, special measures and observation schemes are required to obtain reliable data from plots that are too wide in relation to their length. Border effects are, for example, field or groundwater flow from the plot to a deeper collector drain at one end of the field, or groundwater inflow from an undrained part of the area adjacent to the other end of the field.

26.6 ARRANGEMENT OF PLOTS

The plots should be arranged in such a way that hydrologic interference between subunits is as small as practically possible. A subunit normally comprises four plots and two halves of adjacent buffer plots (see Fig.1). It is defined by the requirement that the drainage conditions of the four individual plots are the same: the same drain depth, drain length, gradient, and drainage materials. A buffer plot separates the subunits. Its width is at least equal to the largest spacing of adjacent plots.

The discharges of Drains 2, 3 and 4 in Subunit A - and the same applies to the other subunits - are preferably measured in one end drain. By doing so the amount of work in measuring and processing is considerably reduced. The discharge of the Drains 1, 5, 6, 10 etc., which border the buffer plots, is usually not measured.

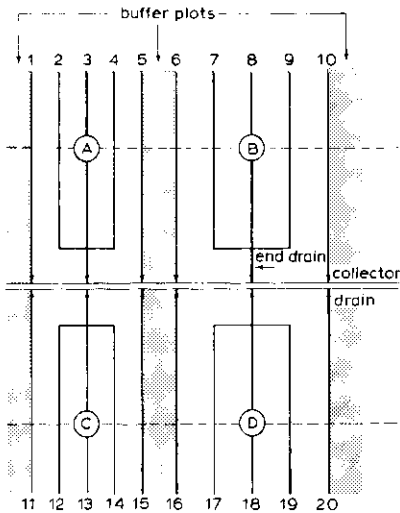


Fig.1. Lay-out of part of an experimental unit consisting of four subunits A,B,C, and D. 1,2,3,4, ... are field drains. Three drains of each subunit are measured in one end-drain

26.7 NETWORK OF OBSERVATION POINTS

Basic observations to be discussed relate to

- discharge of drains
- watertable depth.

It is noted that the quality of drain and irrigation water, soil moisture, root development, soil structure, etc., is not discussed in this chapter. It is, furthermore, assumed that information on factors such as rainfall, windspeed, air temperature, relative humidity and sunshine hours - if relevant for the purpose of the experiment - is being collected in the experimental area or nearby.

26.7.1 DRAIN DISCHARGE

Only the discharge of end-drains is measured (see Fig.1). If all drains flow out individually into the collector, measurements are made of Drains 2-3-4, 7-8-9, 12-13-14, 17-18-19 separately.

Note that the discharge capacity of the collector drain should be large enough to keep its water level below the field drains during periods of high discharge. If the end pipes of field drains are submerged, back pressure will occur in the field drains, whereas discharge measurements will be impossible - unless specific devices are employed.

26.7.2 WATERTABLE DEPTH

Water table observation wells are installed (Fig.2)

- midway between drains to measure the available hydraulic head,
- near one or more of the drains of each subunit, to observe the shape of the water table; wells preferably to be placed at distances of 0.5, 1, 2, 4 and 10 m from the drain,
- at the upper and/or lower ends of some of the subunits, to observe border effects,
- on top of drain tubes, to check the functioning of the drains.

Figure 2 illustrates the placement of wells. If problems of malfunctioning arise, additional wells may be needed.

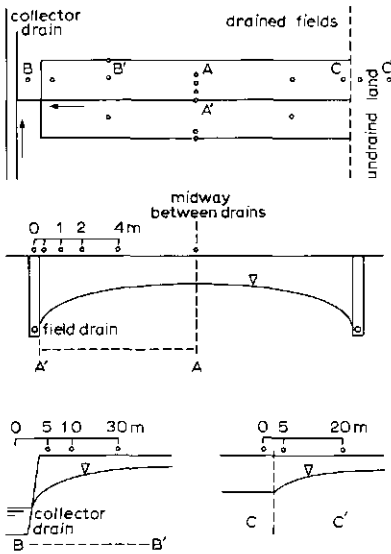


Fig.2. Example of a network of watertable observation wells; o = observation well

Soil hydrological constants

Note that observation wells are open boreholes or boreholes in which perforated pipes have been placed. It is essential that water can seep in and out of the well over its entire saturated depth (Fig.3). Piezometers, i.e. pipes with a short filter at the lower end, may be placed at different depths in cases where a vertical flow of groundwater might be of importance. Depending on the purpose of the experiment, the desired accuracy and the hydrologic heterogeneity of the soil, adaptations in number or type might be useful. For example when the experiment serves to check the effect of different types of drainage materials, more emphasis must be laid on measurements of the loss of hydraulic head in the near vicinity of the drain lines, which makes the use of piezometers more appropriate. Some wells must be measured frequently, others need no longer to be measured once the hydrological influence of the surrounding areas is known (Sect.9).

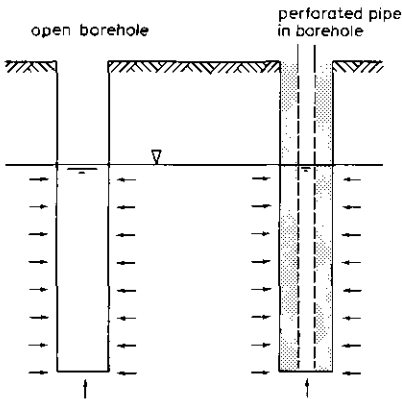


Fig.3. Sketch of watertable observation wells

26.8 MEASURING DEVICES

26.8.1 DRAIN DISCHARGE

Drain discharge may be measured by

- buckets of known volume and using a stopwatch; this method is fairly laborious, especially during periods of heavy rainfall or during and directly after irrigation when several measurements are needed to obtain a true picture of discharge fluctuations. When a quick reaction to rainfall or irrigation is to be expected the difficulty to make observations at night will be another disadvantage. A considerable advantage of the method is its simplicity and low cost,

- discharge recorders attached to drain outlets; several types are available; they offer the advantage that a more reliable picture is obtained of a total or average discharge rate per day or during shorter periods; disadvantages are: very expensive compared to a bucket though often more economic when costs for labour, etc. are taken into account, considerable processing work, though some techniques are available which allow automatic data processing, the requirement that a free-board of 20 cm or more is necessary between drain outlet and the water level in the ditch;

- weirs, etc., disadvantages are 20-30 cm headloss and inaccuracy during periods of low discharge.

It is recommended that at least some recorders are permanently used. They will be helpful in extrapolating data from non-recording systems.

26.8.2 WATERTABLE DEPTH

Watertable depths can be measured in numerous ways, for example by (Chap.21,Vol.III)

- float and tape
- mechanical sounder
- electrical devices (dependent on batteries, sensitive to dirt)
- watertable depth recorders, which are mounted on the observation wells and operate with a float with mechanical registration; some recorders make use of a punched tape.

Also here it is recommended that some automatic recording devices be included to allow correct extrapolation of measurements by other devices.

26.9 FREQUENCY OF OBSERVATIONS

The frequency of observations should be governed by local conditions and the purpose of the trial field. For the experiments discussed in this chapter mostly the relation discharge - hydraulic head is the subject of investigation and the number of observations must be adequate to determine this relation, either for steady or unsteady state conditions. In this respect the drainage intensity of the system under investigation is a significant property, being a characteristic for the speed of the discharge of rain or irrigation water. When a quick discharge occurs after recharge the frequency of observations (of drain discharge and watertable depth) will have to be at least three times a day or more, whereas intervals can gradually be enlarged when the changes in discharge and watertable depth slow down.

26.10 DATA PROCESSING AND ANALYSIS

26.10.1 STEADY STATE CONDITIONS (NON-LAYERED SOIL)

The processing of data obtained from field trials is based on the following equation (Chap.8, Vol.II)

$$q = \frac{8Kdh}{L^2} + \frac{4Kh^2}{L^2} \quad (1)$$

where (see Fig.4)

q = discharge rate (m/day)

h = hydraulic head (m)

K = hydraulic conductivity (m/day)

L = drain spacing (m)

d = "equivalent layer", depending on drain spacing L, distance to impervious base D, and the wet perimeter of the drain $u = \pi r$ (r = radius of the drain). In the value of d the radial resistance near the drain is taken into account.

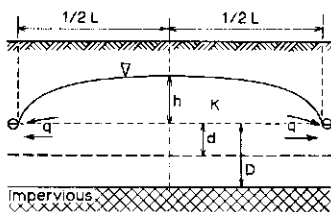


Fig.4. Symbols used in flow equation 1

We can also write Eq.1 as

$$q = Ah + Bh^2 \quad (2)$$

or

$$q/h = A + B \quad (3)$$

where

$$A = \frac{8Kd}{L^2}$$

$$B = \frac{4K}{L^2}$$

The Eqs. 1, 2 and 3 are *steady state* equations which are applicable when, during the experiments, periods can be distinguished during which watertable depth and drain discharge are approximately constant.

Procedure and example (steady state flow)

Procedure

- convert the observed discharge rate into mm or m per day, plot these versus time and draw visually a fitting line through the points (Fig.5A)
- convert the observed watertable depths into hydraulic head values (mm or m), plot these versus time and draw visually a fitting line through the points (Fig.5B)
- plot discharge rates versus corresponding hydraulic heads from both curves and obtain the discharge-head relation (Fig.5C).

Note that Fig.5C can be constructed without the aid of Figs.5A and 5B. The latter, however, are helpful in finding periods during which the hydraulic head and drain discharge are approximately constant and in showing the degree of regularity and accuracy of the measurements. Furthermore, if q and h have not been measured on the same date, interpolations can be made via Figs.5A and 5B.

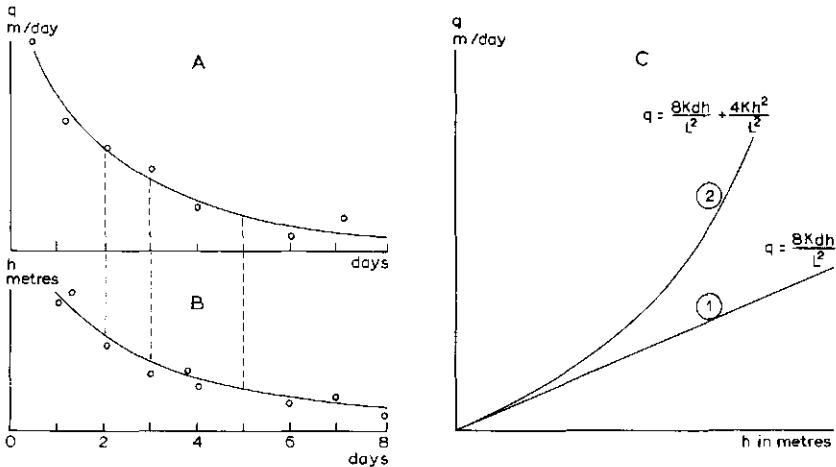


Fig.5. Relation between q and h as derived from q - t and h - t curves

Analysis

Equation 2

$$q = Ah + Bh^2$$

shows that the q - h relation will approach a straight line when the value of Bh^2 is small compared to the value of Ah (Fig.5C, curve 1). Such a straight line points

to a transmissivity of the layers above drain level that is negligible compared to that of the soil layers below the drain level. As a result, the greater part of the drainage water will pass through the layers *below* the drains.

When flow *above* the drain level is not negligibly small, the q-h line will be curved. Its actual shape will depend on the relative contribution made by each of the two parts of the right hand term. The greater the share of the layers above the drains the stronger the curvature will be.

To facilitate the interpretation of the measured q-h relation it may be helpful to plot also q/h versus h (Fig.6). This relation is presented by a straight line making an angle α with the horizontal axis, with (see Eq.3)

$$\tan \alpha = \frac{4K}{L^2}$$

When the value of $\frac{4Kh}{L^2}$ is relatively small the q/h-line will be horizontal.

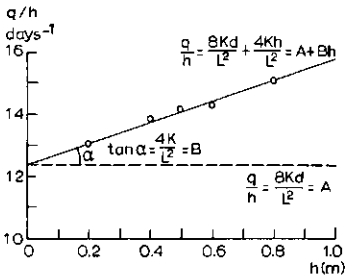


Fig.6. Relation q/h-h, yielding straight lines

Example 1

Consider an experimental field which has been drained by pipes with a radius $r = 0.10$ m, placed at 2 m depth and at a spacing of 100 m. Discharge rate and watertable depth have been measured frequently during periods of little change in watertable position. The observations have been plotted as shown in Figs.5A and 5B and the corresponding values derived are given in Table 1.

Figure 7 shows a plot of q versus h and of q/h versus h. The q-h relation is a slightly curved line which indicates that the greater part of the excess water will flow to the drains through the soil layers below drain level.

The q/h-h relation is a straight line, whose tangent is $B = \frac{4K}{L^2} = 0.4 \times 10^{-3}$ and the value $A = \frac{8Kd}{L^2} = 1.6 \times 10^{-3}$, read from the intersection point on the vertical axis.

With $L = 100$ m, the hydraulic conductivity is found at $K = 1$ m/day and the value for d is 2 m. With the values of L , $u (=0.5$ m) and d known, we can also calculate the depth D if D does not exceed $\frac{1}{4}L$. From Fig.15 we find $D = 2.2$ m. The transmissivity of the layer below drain level is $KD = 1 \times 2.2 = 2.2$ m²/day.

Table 1. Discharge rates and corresponding hydraulic heads based on Fig.5

q (m/day $\times 10^{-3}$)	h (m)	q/h (days ⁻¹ $\times 10^{-3}$)
4.23	1.8	2.35
3.60	1.6	2.25
3.00	1.4	2.14
2.52	1.2	2.10
2.00	1.0	2.00
1.53	0.8	1.91
1.10	0.6	1.83
0.70	0.4	1.75
0.33	0.2	1.65

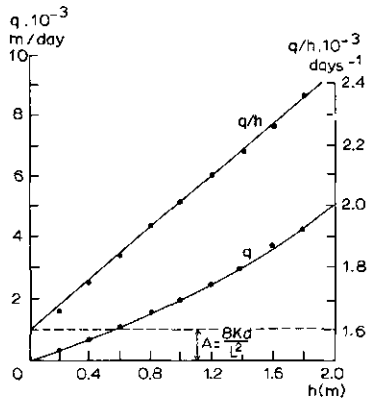


Fig.7. Plots of q versus h and q/h versus h in the calculation of K and Kd

26.10.2 STEADY STATE CONDITIONS (LAYERED SOIL)

Till now it has been assumed that the soil is homogeneous and isotropic. If the soil profile consists of two layers of distinct different hydraulic conductivity Eq.1 is still applicable if the boundary of the two layers is at the level of the drains.

Equation 1 is then written as

$$q = \frac{8K_2dh}{L^2} + \frac{4K_1h^2}{L^2} \quad (4)$$

where K_1 and K_2 are the hydraulic conductivities of soil above and below the drain level respectively. Processing of the data as described above now yields K_1 and K_2d .

It is noted that the curvature of the lines representing the q - h relations will be more pronounced as the K_1/K_2 ratio increases.

If the boundary of the two layers is located above the drain level we get q/h - h lines of the type of Fig.8. Line 1 shows upwards of $h = h_1$ a sharp increase in slope, indicating a layer with relatively high hydraulic conductivity commencing roughly at h_1 ; Line 2 indicates that a layer of relatively poor hydraulic conductivity is found beyond h_2 . This situation with a contribution of flow through two layers with their boundary above drain level can be expressed as follows

$$q/h = \frac{8K_2d}{L^2} + \frac{4K_2h_z}{L^2} + \frac{4K_1}{L^2} (h-h_z) \quad (5)$$

where h_z denotes the height of the transition zone above drain level.

If the boundary of the two layers is located below the drain level, Eq.1 is not applicable. Different expressions are to be used instead, such as those developed by ERNST (Chap.8, Vol.II).

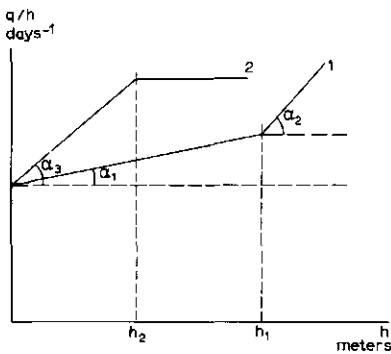


Fig.8. Relation q/h - h in a two-layer profile

26.10.3 UNSTEADY STATE CONDITIONS

The above analysis, with the aid of steady state flow equations, only applies when during experimentation, periods can be distinguished during which the watertable depth is approximately constant. When the phreatic reservoir is recharged by irrigation losses or leaching water, the shape of the water table near the drains is different from that during steady state conditions or during the recession following the cessation of the recharge conditions. During the period the water table rises the head near the drains and consequently the discharge rate are greater than during recession (border or shoulder effect, Fig.9). The discharge-head relation will therefore differ for conditions of a rising water table, a steady state flow, as well as a falling water table. Quite different formulas describe the relation for the latter two conditions. During the first situation, when the water table is rising, no constant discharge-head relation can be found.

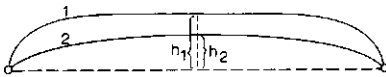


Fig.9. Shape of water table: 1. during recharge (shoulder effect), 2. during tail recession

For computation of the discharge and the hydraulic head use can be made of the formulas derived by KRAIJENHOFF VAN DE LEUR (Chap.8, Vol.II).

$$q(t) = \frac{8R}{\pi^2} \left[\sum_{n=1,3,5}^{\infty} \frac{1}{n^2} (1 - e^{-n^2 t/j}) - \sum_{n=1,3,5}^{\infty} \frac{1}{n^2} (1 - e^{-n^2 (t-t_r)/j}) \right] \quad (6)$$

$$h(t) = \frac{4Rj}{\pi\mu} \left[\sum_{n=1,-3,5}^{\infty} \frac{1}{n^3} (1 - e^{-n^2 t/j}) - \sum_{n=1,-3,5}^{\infty} \frac{1}{n^3} (1 - e^{-n^2 (t-t_r)/j}) \right] \quad (7)$$

where

- $q(t)$ = discharge rate (m/day)
- $h(t)$ = available hydraulic head (m)
- R = recharge rate (m/day)
- t_r = period of steady recharge (days)
- t = time (starting from the beginning of the recharge) (days)
- μ = effective porosity (dimensionless)
- $j = 1/\alpha = \frac{\mu L^2}{\pi^2 KD}$ = groundwater reservoir-coefficient (days)
- K = hydraulic conductivity (m/day)
- D = distance to impervious base (m)
- L = drain spacing (m)

Soil hydrological constants

At a certain time $t \approx 0.4 j$, after cessation of the recharge, the second and further terms of the infinite series of Eqs.6 and 7 become very small and are therefore negligible ("tail recession"). Thus Eqs.6 and 7 reduce to

$$q(t) = \frac{8R}{\pi^2} \left[(1-e^{-t/j}) - (1-e^{-(t-t_r)/j}) \right] \quad (8)$$

$$h(t) = \frac{4Rj}{\pi\mu} \left[(1-e^{-t/j}) - (1-e^{-(t-t_r)/j}) \right] \quad (9)$$

Substituting two values $t = t_1$ and $t = t_2$ (t_1 and $t_2 > t_r + 0.4j$ (see Fig.10)) one obtains

$$q(t_2) = q(t_1) e^{-(t_2-t_1)/j} \quad \text{or} \quad (t_2-t_1)/j = 2.3 \log \frac{q(t_1)}{q(t_2)} \quad (10)$$

$$h(t_2) = h(t_1) e^{-(t_2-t_1)/j} \quad \text{or} \quad (t_2-t_1)/j = 2.3 \log \frac{h(t_1)}{h(t_2)} \quad (11)$$

Combining Eq.8 and Eq.9 gives

$$q(t) = \frac{2\mu}{\pi j} h(t) \quad (12)$$

or, since

$$j = \frac{\mu L^2}{\pi^2 KD} \quad (13)$$

$$q(t) = \frac{2\pi KD}{L^2} h(t) \quad (14)$$

Note that

- the groundwater reservoir-coefficient j may, according to Eqs.10 and 11, be calculated either from the hydraulic heads, or from the discharge rates; when using equations for steady state flow both hydraulic head and discharge should be known,

- strictly speaking, the numerical value of D in Eq.13 is equal to the thickness of the aquifer below the drains plus $1/4 \{h(t_1) + h(t_2)\}$. To apply the equation for unsteady state flow, however, D should be constant. In practice this is considered to be so if the transmissivity of the part of the phreatic aquifer below the drains exceeds by far that of the part above the drains,

- to include the radial resistance near the drain the D -value may be replaced by Hooghoudt's d -value.

Equation 14 may then be written as

$$q(t) = \frac{2\pi Kd}{L^2} h(t) \quad (15)$$

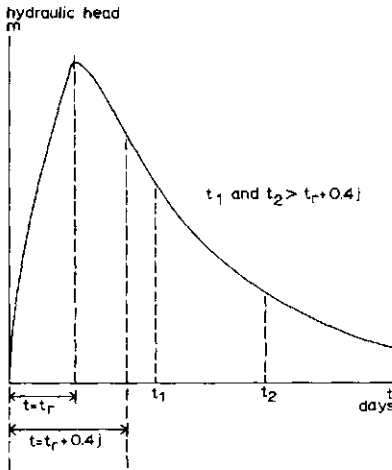


Fig.10. Relation between $h(t)$ and t , during recharge and tail recession

Procedure and example (unsteady state flow)

Example 2

The procedure and analysis is demonstrated by the following example.

Consider an experimental field that has been drained by pipes at a spacing of 30 m. The pipes with a radius $r = 0.10$ m have been placed at a depth of 1.80 m (Fig.11).

Soil investigations show a thick layer of clay with a plastic consistence whose upper boundary is at a depth of 4.80 m below ground surface. From hydraulic conductivity measurements and additional observations on the seasonal fluctuations of the water table it is concluded that the transmissivity of this layer is very small compared to that of the overlying soil and may be considered an impervious floor.

On a certain day 140 mm of water is applied, of which 40 mm percolate below the rootzone. It is assumed that all of it recharges the phreatic aquifer on that same day. During the day of recharge and the following days the watertable depth and the discharge rate are measured several times a day.

Soil hydrological constants

The experiment will serve - amongst other things - to find the coefficient (j) and/or to collect basic information on such individual physical soil characteristics as hydraulic conductivity (K), transmissivity (KD), and effective porosity (μ).

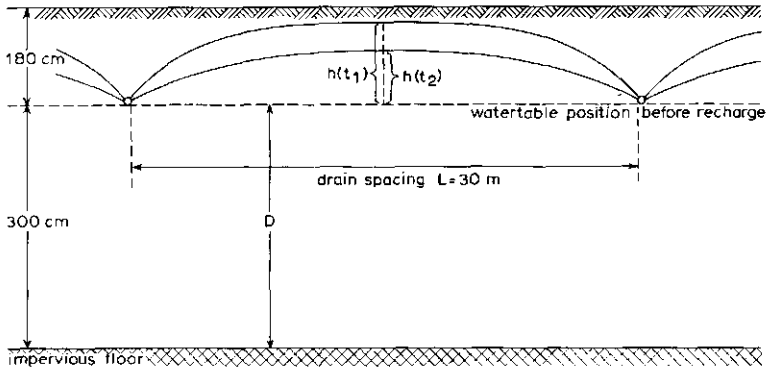


Fig.11. Drainage conditions of Example 2 (unsteady flow)

Calculation of the groundwater reservoir-coefficient

To arrange the field observations and calculate the groundwater reservoir-coefficient, proceed as follows

- convert the observed discharge rates into mm or m per day, plot these versus time and draw visually a fitting line through the points (Fig.12),
- convert the observed watertable depths into hydraulic head values (mm or m), plot these versus time and draw visually a fitting line through the points (Fig.12),
- read from the two graphs the corresponding values at the end of the days and compose Table 2.

Table 2. Recharge, discharge rates and corresponding hydraulic heads based on Fig.12

t	=	1	2	3	4	5	6	7	8	days
R	=	40	-	-	-	-	-	-	-	mm
q(t)	=	14.4	5.9	4.4	3.4	2.6	2.0	1.6	1.2	mm/day
h(t)	=	495	430	340	265	205	160	125	100	mm

- plot $q(t)$ and/or $h(t)$ values from this table versus time on semi-log paper and obtain the lines of Fig.13,

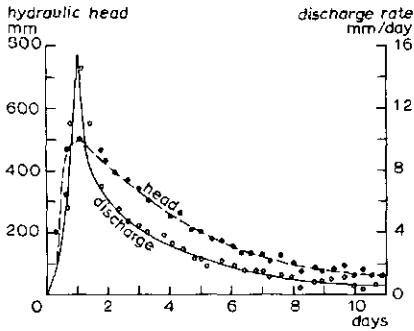


Fig.12. Watertable position and discharge rates observed and converted into hydraulic heads (mm) and discharge rates (mm/day).

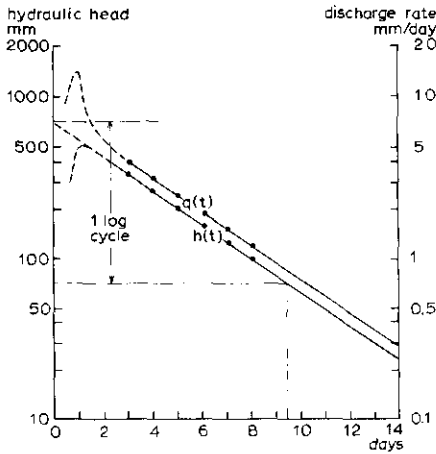


Fig.13. Plots of $q(t)$ versus time and $h(t)$ versus time. Data taken from Table 2

Note that, according to Eqs.10 and 11 - which apply to tail recession - these lines should be straight and parallel to one another.

- calculate the groundwater reservoir-coefficient (j).

A practical way of calculation is by using the Eqs.10 and 11

$$1/j = \frac{2.3 \{ \log h(t_1) - \log h(t_2) \}}{t_2 - t_1}$$

and

$$1/j = \frac{2.3 \{ \log q(t_1) - \log q(t_2) \}}{t_2 - t_1}$$

In both cases there results

$$1/j = 2.3 \tan \alpha$$

Observe that $h(t_1)$, $q(t_1)$ and $h(t_2)$, $q(t_2)$ are points of the straight part of the lines. They can be selected freely, taking into account that $h(t_1)$, $q(t_1)$ presents an earlier date than $h(t_2)$, $q(t_2)$.

To obtain $\tan \alpha$ it is practical to select one full logarithmic cycle on the h or q axis, e.g. from 700 to 70 ($\log 700 - \log 70 = \log \frac{700}{70} = \log 10 = 1$). The value of $\tan \alpha$ is then found from

$$\tan \alpha = \frac{1}{t_2 - t_1}$$

It appears from Fig.13 that

$$\tan \alpha = \frac{1}{9.5} = 0.105$$

and therefore

$$j = \frac{1}{2.3 \times 0.105} = 4.1 \text{ days}$$

Note that the lines of Fig.13 become straight at the time $t \approx 0.4j$ days after the recharge. The value of t cannot be calculated at the time the straight line pieces must be drawn, since j is then still unknown. In the case of Fig.13 this does not present a problem since the position and direction of the straight part is clear from the points obtained between the third and eighth day. It often happens, however, that the observations appear somewhat scattered in the lower region of the lines where discharge rates are low and water tables move slowly. The inaccuracy of the observations may then have a considerable impact.

The uncertainty about the beginning and the end of the straight part causes the need for frequent and accurate observations during, say, the period between the second and the sixth day after water application. Since $j = 4.1$ days, it follows that $t \approx 0.4 \times 4.1 \approx 1.7$ days. Thus in Fig.13, the line will be straight from $t \approx 3$ days onwards.

Calculation of the hydraulic conductivity and transmissivity

To calculate the hydraulic conductivity K , plot $q(t)$ versus $h(t)$ values from Table 2 and find $\frac{q}{h} = 0.0127$ (see Fig.14). It is recalled from Sect. 10.1 that the q/h relationship yields a straight line when most of the water passes to the drains through the soil below the drain level.

The variations in watertable position will then have only a minor effect on the actual thickness of the phreatic aquifer (D) and the transient flow equations are applicable.

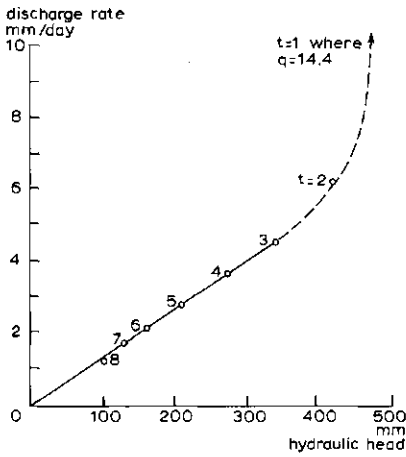


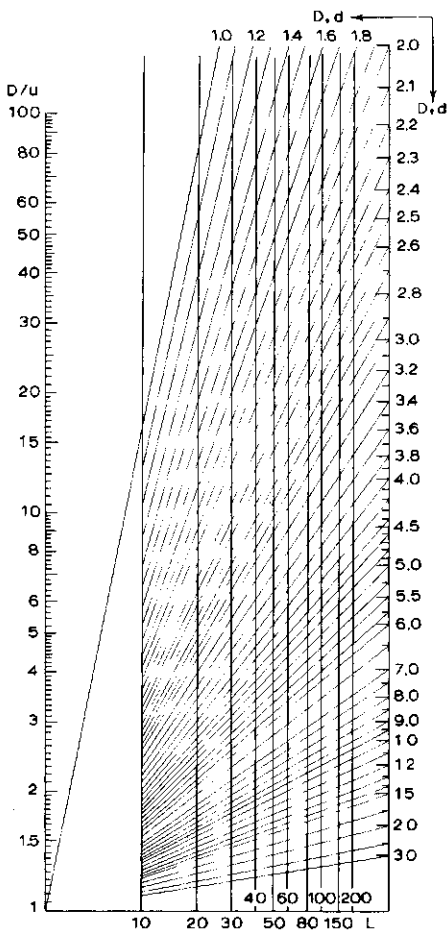
Fig.14. Discharge rate versus hydraulic head. Data taken from Table 2

Applying Eq.15 which after transposing reads

$$Kd = \frac{q}{h} \frac{L^2}{2\pi}$$

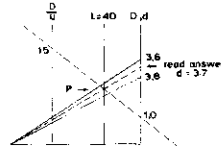
there results, with $L^2 = 900$, $Kd = 1.8 \text{ m}^2/\text{day}$. To obtain the hydraulic conductivity K from the Kd -value find Hooghoudt's d -value from tables or graphs. For $r = 0.10 \text{ m}$ ($u = \pi r$), $L = 30 \text{ m}$ and $D = 3 \text{ m}$ (see Fig.15), there results $d = 1.97 \text{ m}$ and consequently, $K \approx 0.9 \text{ m/day}$.

The transmissivity $KD = 0.9 \times 3 = 2.7 \text{ m}^2/\text{day}$.



Nomograph for the calculation of Hooghoudt's d-value according to

$$d = \frac{D}{\frac{8D}{\pi L} \ln \frac{D}{u} + 1}$$



To use:

1. Select appropriate values for D/u and D
2. Connect selected D/u on the left hand scale with D on the right hand scale
3. Find point P where connection line and selected L-scale intersect
4. Read value of P on the right hand D,d-scale as Hooghoudt's d-value

Example

If D/u = 15 and D = 10 m, then with L = 40 m, read d = 3.7

Note

If D < 2 use ERNST or calculate d with the above formula

If D > 1/2 L use D = 1/2 L

Fig.15. Nomograph for the determination of the equivalent depth d. After VAN BEERS (unpublished)

Calculation of the effective porosity

The effective porosity μ may be calculated from

$$\mu = \frac{\pi^2 K d_j}{L^2} = 0.08$$

or from Eq.12

$$\mu = \frac{q \pi j}{2h} = 0.08$$

or - more roughly - directly from Table 2 or Fig.12 as the change in hydraulic head during the first day (495 mm) under influence of the recharge of 40 mm

$$\mu = \frac{40}{495} = 0.08$$

The effective porosity may also be found from the volume (W) of water released by the soil when the water table drops from position $h(t_1)$ to position $h(t_2)$ in a known time interval during the tail recession, according to the expression $W \approx 0.7\mu \{h(t_1) - h(t_2)\}$ where W is calculated from the measured discharge rates.

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PRINCIPAL SYMBOLS USED IN VOLUME III

<i>Symbol</i>	<i>Description</i>	<i>Dimension</i>
A	cross-sectional area	L^2
	energy rate used for heating the air (sensible heat)	$\text{cal } L^{-2}T^{-1}$
a	empirical constant	dimensionless
b	empirical constant	dimensionless
C	geometry factor, correction factor	dimensionless
Cap	capillary rise	LT^{-1}
C_1^{BC}	crop coefficient of Blaney and Criddle	dimensionless
c	hydraulic resistance of semipervious layer against vertical flow	T
	Euler constant $c = 0.57722\dots$, empirical constant	dimensionless
D	thickness of layer	L
	deep drainage from the rootzone	LT^{-1}
d	thickness of equivalent depth in Hooghoudt's formula, grain diameter, equivalent diameter of pore	L
E	evaporation of a free water surface, evapotranspiration	LT^{-1}
E_a	actual evapotranspiration	LT^{-1}
E_p	potential evapotranspiration	LT^{-1}
E_x	isothermal evaporation	LT^{-1}
EC	electrical conductivity	$\text{ohm}^{-1}\text{cm}^{-1}$
Est	estimate	-
e	vapour pressure	L
	base of natural (Napierian) logarithm $e = 2.718\dots$	dimensionless
F	frequency of occurrence	dimensionless
f	empirical conversion factor	dimensionless
$f(u)$	wind velocity function	LT^{-1}
$f(x)$	normal probability density function of x	dimensionless
G	rate at which heat is stored in soil, water and vegetation	$\text{cal } L^{-2}T^{-1}$
g	acceleration due to gravity	LT^{-2}
H	depth of the borehole below the initial groundwater table	L

<i>Symbol</i>	<i>Description</i>	<i>Dimension</i>
H_{lo}^n	net outgoing long-wave radiation	cal L ⁻² T ⁻¹
H_o	net radiation	cal L ⁻² T ⁻¹
H_{sh}	total incoming short-wave radiation	cal L ⁻² T ⁻¹
H_{sh}^{top}	extra-terrestrial radiation	cal L ⁻² T ⁻¹
h	hydraulic head	L
	relative humidity	dimensionless
I	annual heat index	-
	infiltration rate, irrigation water supplied	LT ⁻¹
i	hydraulic gradient, serial number	dimensionless
	monthly heat index	-
j	groundwater reservoir coefficient	T
K	hydraulic conductivity	LT ⁻¹
K_h	capillary conductivity	LT ⁻¹
$K_o(x)$	modified Bessel function of the second kind and zero order (Hankel function)	dimensionless
KD	transmissivity of the aquifer	L ² T ⁻¹
$(KDc)^{\frac{1}{2}}$	leakage factor of semipervious layer	L
k	function of soil moisture content, total number of intervals	dimensionless
L	length, drain spacing	L
	latent heat of evaporation L = 59 (cal/0.1 cm ³)	cal L ⁻³
LE	energy rate used for evapotranspiration	cal L ⁻² T ⁻¹
L^{Tc}	evaporative demand of the atmosphere	LT ⁻¹
M	miscellaneous energy terms	cal L ⁻² T ⁻¹
	molecular weight	M mole ⁻¹
m	number of data in an interval	dimensionless
N	maximum possible duration of bright sunshine hours	T
	maximum number of data in a sample	dimensionless
n	actual duration of bright sunshine hours	T
	number, empirical constant	dimensionless
P	rainfall	LT ⁻¹
$Perc$	percolation	LT ⁻¹
$Prob$	probability of occurrence	-

Symbols

Symbol	Description	Dimension
p	probability of exceedance	dimensionless
p_{BC}	monthly percentage of daylight hours in the year	dimensionless
pF	logarithm of the water tension in cm water column	dimensionless
Q	discharge	L^3T^{-1}
q	discharge per unit surface area	LT^{-1}
	discharge per unit length or per unit width	L^2T^{-1}
	probability of non-exceedance	dimensionless
R	effective recharge, runoff	LT^{-1}
	universal gas constant	$ML^2T^{-2} \text{ mole}^{-1} \text{ } ^\circ K^{-1}$
r	radius, radial distance from the centre of a well	L
	correlation coefficient, rank number	dimensionless
S	storage coefficient	dimensionless
	depth of impervious layer below bottom of borehole, storage, tension	L
SAR	sodium adsorption ratio	dimensionless
s	standard error	dimensionless
T	air temperature	$^\circ C$
	absolute temperature	$^\circ K$
	return period	T
t	time	T
U	relative humidity	dimensionless
	specific surface	L^{-1}
u	mode of the Gumbel distribution	dimensionless
	wind velocity	LT^{-1}
	wetted perimeter of drain	L
v	flow velocity, infiltration rate	LT^{-1}
W	weight	M
	soil moisture content	L
W(u)	Theiss's well function	dimensionless
w	radial resistance	$L^{-1}T$
x, y	normal variates	dimensionless
x, y, z	Cartesian coordinates	dimensionless

<i>Symbol</i>	<i>Description</i>	<i>Dimension</i>
α	angle	degree
	regression coefficient, standard deviation of the Gumbel distribution	dimensionless
β	Bowen ratio	dimensionless
γ	psychrometric constant $\gamma = 0.66 \text{ mbar}/^{\circ}\text{C}$ or $0.485 \text{ mm Hg}/^{\circ}\text{C}$	$\text{L } ^{\circ}\text{C}^{-1}$
δ	partial derivative sign	dimensionless
Δ	small increment of	dimensionless
	slope of saturation vapour pressure - temperature curve	$\text{L } ^{\circ}\text{C}^{-1}$
ϵ	porosity, median	dimensionless
η	dynamic viscosity	$\text{ML}^{-1}\text{T}^{-1}$
μ	effective porosity, storage coefficient, drainable pore space, mathematical expectation of the normal variate	dimensionless
ρ	mass density	ML^{-3}
σ	surface tension of water against air	MT^{-2}
	standard deviation	dimensionless
σ^2	variance of the normal variate	dimensionless
ϕ	latitude	degree
	suction	L
!	factorial	dimensionless

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E R R A T A T O V O L U M E I I (DRAINAGE PRINCIPLES AND APPLICATIONS)

Chapter 8

Pag.11, Figure 4: turn 180 degrees

Pag.13, Figure 6: caption: if $\frac{L}{h} < 100$, read: if $\frac{L}{h} > 100$

Pag.14, Figure 7: caption: if $\frac{L}{h} > 100$, read: $\frac{L}{h} < 100$

Pag.20, 29th line: $\frac{1}{2}(y + h)$, read: $y + \frac{1}{2}h$

Pag.27, bottom line: Since $D_o = 0.8$ m the condition $D_o < \frac{1}{4} L$ is fulfilled.

Read: Since $D_o = 0.8$ m the conditions $D_o < \frac{1}{4} L$ (radial flow)

and $D_1 + D_2 < \frac{1}{4} L$ (horizontal flow) are fulfilled.

Pag.28, 12th and 19th lines: 1.125 m = ... and 1.25 m = ... : delete m.

26th line: $h = \frac{qL^2}{8KD}$, read: $h = \frac{qL^2}{8Kd}$

Pag.29, Figure 4: The user of this nomograph is referred to Chapter 26 in Volume III where in Figure 15 the same nomograph is depicted with more explanation.

Pag.30, Add to caption of Figure 15: Family of curves for different values of u ($u = \pi r_o$).

Pag.32, Equation 33: under the sigma sign: $n = 1, -3, 5$, read: 1,3,5.

Equation 34: $\alpha = \frac{\pi^2 KD}{\mu L}$, read: $\alpha = \frac{\pi^2 KD}{\mu L^2}$.

Pag.34, 9th line: ... is only required in ... : delete the word "only".

Pag.35, 3rd line: $m = 0.05$, read: $\mu = 0.05$

Pag.36, Equation 45: $q_t = \frac{8}{\pi^2} \alpha R_i \sum_{n=1,3,5}^{\infty} e^{-n^2 \alpha t}$

read: $q_t = \frac{8}{\pi^2} \alpha R_i \sum_{n=1,3,5}^{\infty} e^{-n^2 \alpha t}$

Pag.40, Table 3, 8th column g_t , 19th line: 0.990, read: 0.890.

Pag.41, bottom line: $h = h_1 = \frac{R}{\mu} j c_1$, read: $h = h_1 = \frac{R}{\mu} j c_1$

Pag.43, 8th line, 3rd column: c/t , read: c_t

Chapter 9

Pag.69, penultimate line: $w_e \approx w_{fc}$, read: $w_e \approx 2 w_{fc}$

Chapter 10

Pag.107, 6th and 7th lines: ... e_a , (being the ratio between the quantity reaching the fields and the evapotranspiration of the crop), ...

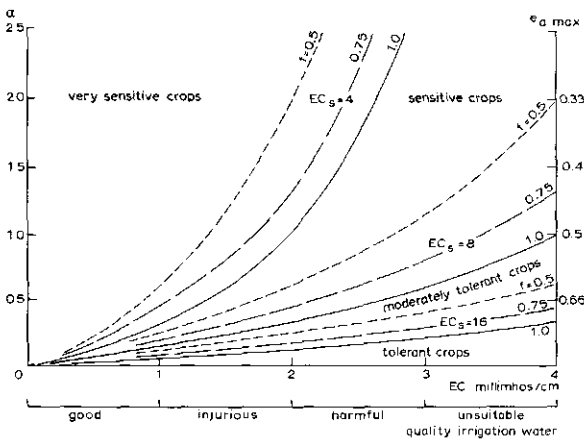
read: e_a , (being the ratio between the evapotranspiration of the crop and the quantity reaching the fields), ...

Pag.122, 11th line from bottom: EC_s , read: EC_e

Equation 18: $EC_{fc} = 2 EC_s$, read: $EC_{fc} = 2 EC_e$

8th line from bottom: EC_s , read EC_e

Pag.123, Replace Figure 11 by Figure below



Pag.123, 7th and 11th lines from bottom: EC_s -values and value of EC_s ,
read: EC_e -values, and value of EC_e

Chapter 11

Pag.140, Figure 3: values along abscissa: 10, 20, 30, 40, 50, and 60
read: 1.0, 2.0, 3.0, 4.0, 5.0, and 6.0

Pag.162, Table 6, Heading, 3rd Column: R_i/μ (mm) (3)
read: R_i/μ (m) (3)

Chapter 12

- Pag.173, Heading of Table I, 2nd line: Bessel function of the first kind
read: Bessel function of the second kind
- Pag.176, 1st line below Equation 14: where K_0 is a modified Bessel function
(Hankel function) of the first kind ...
read: . . . of the second kind . . .
- Pag.184, 6th and 9th lines: a bracket should be placed between 2.3 and
 $\log \frac{2.25KD}{r^2S}$ n and one behind the last term on the 9th line

Chapter 13

- Pag.195, 1st line: Substituting this form into Eq.7 yields $a = \pm 1/\sqrt{KDc}$
read: ... yields $a = \pm 1/\sqrt{KDc}$
- Pag.200, Eq.24: $Q = 2\pi\beta r [-C_1 I_1(r/\lambda) - C_2 K_1(r/\lambda)]$
read: $Q = 2\pi\beta r [-C_1 I_1(r/\lambda) + C_2 K_1(r/\lambda)]$
- Pag.202, Equation 32: behind equal sign, $2\pi\beta r$, read: $2\pi\beta R$
- Pag.208, middle of page: $\frac{KD}{\mu} = \frac{x}{2\sqrt{t}} \frac{1}{u} = \frac{4}{2} \times \frac{1}{0.1} = 20$
read: $\sqrt{\frac{KD}{\mu}} = \frac{x}{2\sqrt{t}} \frac{1}{u} = \frac{4}{2} \times \frac{1}{0.1} = 20$
- Pag.219, Example 8, 1st and 2nd lines: Assume a ditch with top width $B = 3$ m,
bottom width $b = 1$ m, water depth $y_o = 1$ m, and $h_o = 4$ m above ...
read: Assume a ditch with top width $B = 3$ m, bottom width $b = 1$ m,
water depth 1 m, and an impervious layer 4 m below the bottom of the
ditch (hence $y_o = 1 + 4 = 5$ m).
- Pag.220, 1st line: $B/u = 3/3.82 = 0.76$ m, read: $B/u = 3/3.82 = 0.76$
5th line: $h_1/y_o = 4.8/5 = 0.96$ m, read: $h_1/y_o = 4.8/5 = 0.96$
6th line: $(B/u > 0.9)f = 1.08$, read: $(B/u < 0.9)f = 1.08$

Chapter 14

- Pag.228, Equation 1: $\frac{\delta^2 h}{\delta x^2} \frac{\delta^2 h}{\delta y^2} = 0$, read: $\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} = 0$

Pag.229, 8th line: Equation 1 is linear ..., read: Equation 2 is linear.

Pag.231, Equation 7: denominator $\ln(2.25 \text{ KD}/L\mu r_w)$
read: $\ln(2.25 \text{ KDt}/L\mu r_w)$

Pag.241, Equation 22: $h_e - h_w = \frac{Q_o}{2\pi \text{KD}} \ln (r/r_e) - 1/2$
read: $h_e - h_w = \frac{Q_o}{2\pi \text{KD}} [\ln (r/r_e) - 1/2]$

Chapter 15

Pag.250, 1st line: soils, runoff may not occur at all, ...
read: soils, overland flow may not occur at all, ...

Pag.256, 4th line: v = wave celerity, read: v_w = wave celerity

Pag.261, 1st line of Section 3.2: The vertical in the diagram of Fig.14 represents the available physical formation on ...

read: The vertical in the diagram of Fig.14 represents the available physical information on ...

Equation 3: $Q = \frac{g}{\nu} \frac{\pi}{128} d^2 \frac{\Delta h}{L}$ read: $Q = \frac{g}{\nu} \frac{\pi}{128} d^4 \frac{\Delta h}{L}$

Pag.262, Equations 6 and 7 and Fig.15: q , dq , and $q(t)$, read: Q , dQ , $Q(t)$

Pag.278, 7th line from bottom: Here the same great depth of rain is supposed to fall over a period of duration T_r ,
read: T_R in stead of T_r

Pag.279, Figure 23, bottom left-hand corner:

> 150 hrs (T_r) read: > 150 hrs (T_R)
rainfall duration rainfall duration