PROCEEDINGS OF THE INTERNATIONAL DRAINAGE WORKSHOP

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PROCEEDINGS OF THE INTERNATIONAL DRAINAGE WORKSHOP

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Edited by
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PREFACE

In 1976 the Government's Service for Land and Water Use launched the idea of bringing together a number of drainage specialists from different countries to exchange information on, and to discuss problems of, research, design, and installation of agricultural drainage systems in humid areas.

After the idea had been discussed with other institutions in The Netherlands, the decision was made to organize an International Drainage Workshop in Wageningen.

The organizing institutions, namely:

The Government Service for Land and Water Use (LD)
The Institute for Land and Water Management Research (ICW)
The International Institute for Land Reclamation and Improvement (ILRI)
The International Agricultural Centre (IAC)

nominated an organizing committee consisting of:

T.Couwenhoven (LD)	*chairman*
J.A.C.Knops (ILRI)	*secretary*
J.Wesseling (ICW)	*member*
H.E.van Dissel (IAC)	*member*

The organizing committee decided that the Workshop should be held from May 16-21, 1978, the week before the ICID Congress in Athens (Greece), because it was expected that many of the people who would be invited to the Workshop would also be planning to attend that meeting.

It was originally intended that four problem areas would be discussed at the Workshop. Because of the interests of the organizing institutions, it was later decided to add the drainage problems of irrigated areas. Unfortunately the subject of Maintenance and Economics of Drainage had to be cancelled because of the lack of discussion papers, although the subject was discussed incidentally under the other headings. Finally, the following problem groups were distinguished:

100 Design and Research
200 Materials
300 Installation Methods
400 Drainage of Irrigated Lands

In total 26 participants from abroad and 22 from The Netherlands responded to the invitation to attend the Workshop and to provide discussion papers.

The organizing committee was able to distribute these papers among the participants well before the start of the Workshop. This made it unnecessary to present the papers during the Workshop so that the time could be devoted entirely to discussions. The Workshop spent 2 days on group discussions, 1 day on an excursion and 1 day on a plenary session. In the latter session the preliminary conclusions from the various groups were discussed.

When the Workshop was originally being planned, it was not intended to publish the papers. ILRI planned to publish an overall review of the papers and the main results of the discussions. An editorial committee was given the responsibility for this publication. Its members were:

Chief editor:	*J.Wesseling (ICW)*
Group 100:	*R.J.Oosterbaan (ILRI) and G.P.Wind (ICW)*
Group 200:	*J.A.C.Knops (ILRI) and C.L.van Someren (ILRI)*
Group 300:	*D.Boels (ICW) and W.H.Naarding (LD)*
Group 400:	*R.van Aart (ILRI) and J.H.Boumans (Euroconsult)*

During the plenary session of the Workshop the majority of the participants were in favour of publishing the papers in full. The organizing committee found ILRI willing to comply with these wishes. Several papers submitted to the Workshop, however, were purely discussion papers and not suitable for publication. Other papers had been submitted for publication elsewhere or had already been published. Therefore, the organizing committee had to set the following rules:

- All papers published elsewhere, only the title, reference, and an abstract would be taken up;
- Participants had to decide personally on the publication of their paper either in the submitted form or after revision;
- Papers submitted, but not intended for publication, would nevertheless be used by the editorial committee for their conclusions.

Following these rules, not all the papers submitted to the Workshop are found in this book. Although the decision to publish the papers has delayed publication somewhat, the editorial committee is glad that in this way the wishes of the majority of the participants could be met.

The report of the Workshop now consists of two sections. Section one, produced by the editorial committee mentioned above, gives a brief review of papers, discussions, and conclusions of the various groups. As agreed upon during the Workshop, a draft of this section was sent to the participants for their comments.

In the final version three types of references are given. References with a name followed by a year, e.g. Johnston (1978) refer to an article in the literature. If a name is followed by a number e.g. Johnston (4.01) the reference pertains to a paper in the second part of the book, namely Group 400, Paper 01. If only a name is given, the reference pertains to an opinion, remark, or point stated by a participant during the Workshop.

Section II contains the papers, which have been divided into the groups mentioned above. One should be aware that some of the papers were discussed in more than one group.

The sequence of the papers is arbitrary, in the sense that the numbering in each group was determined mainly by the order in which they appeared in the editorial procedure.

The reader will notice the variation in style in the papers presented. The editor, however, did not feel that it was his task to superimpose uniformity.

The editorial committee is grateful to all participants for their comments on the first draft of Section I and the cooperation they gave in revising their papers. It appreciates the willingness of ILRI provide funds for the publication and to go through all the problems a publication of this kind brings with it.

It is hoped that the book will find its way, not only among those who participated in the Workshop, but also among the numerous other people in the world who are interested in the problems of agricultural drainage.

J. Wesseling,
Wageningen, December 1978

AGENDA AND PROGRAMME

Tuesday, May 16

09.00 - 10.30	Registration
10.30 - 12.30	Group discussions, 1st session
13.45 - 15.30	Group discussions, 2nd session
16.00 - 17.45	Group discussions, 3rd session
20.00 - 23.00	Official opening
	Welcome address by Mr.T.Couwenhoven, Chairman of the Organizing Committee
	Welcome address by Mr.J.Koopman, Director of the Government Service for Land and Water Use (LD)
	Speech by Mr.W.A.Segeren, Head of the Research Department (LD)
	Film and social evening

Wednesday, May 17

08.30 - 10.15	Group discussions, 4th session
10.45 - 12.30	Group discussions, 5th session
13.45 - 15.30	Group discussions, 6th session
16.00 - 17.00	Group discussions, 7th session

Thursday, May 18

08.00 - 21.00	Excursions
	Morning: Visit to IJsselmeerpolders Development Authority, Lelystad
	Afternoon: visit to Land Consolidation project "Het Grootslag", Bovenkarspel
	The Group secretaries used this day for preparation of group reports

Friday, May 19

08.30 - 10.15	Group discussions, formulation of conclusions and recommendations
10.45 - 12.30	1st plenary meeting, presentation of group conclusions and recommendations by their respective chairmen
13.45 - 15.30	2nd plenary meeting, adjusting the group conclusions and recommendations
16.00 - 17.00	3rd plenary session, final conclusions and recommendations
	Closing session

GROUP SCHEDULES

Group 100	*Mr.J.Wesseling, chairman* *Mr.R.J.Oosterbaan, secretary*	
	Session 1-2:	Theory of groundwater flow, surveys
	Session 3-4:	Drainage criteria
	Session 5-6:	Regional drainage problems
	Session 7:	Research needs and development
Group 200	*Mr.W.J.Ochs, chairman* *Mr.J.A.C.Knops, secretary*	
	Session 1:	General
	Session 2:	Research
	Session 3:	Entrance resistance
	Session 4:	Hydrodynamics, Soil stabilizers
	Session 5:	Hydraulics in pipes
	Session 6:	Maintenance
	Session 7:	Iron complexation
Group 300	*Mr.G.Spoor, chairman* *Mr.D.Boels, secretary*	
	Session 1:	Theory of mechanics
	Session 2-3:	Installation methods
	Session 4-5:	Experiences with drain performance
	Session 6-7:	Precision and control
Group 400	*Mr.J.van Schilfgaarde, chairman* *Mr.R.van Aart, secretary*	
	Session 1:	Theory of drainage and leaching
	Session 2-3:	Criteria
	Session 4:	Irrigation-drainage relations
	Session 5:	Drainage systems and techniques
	Session 6-7:	Economic evaluation, Research needs
Plenary sessions	*Mr.J.Wesseling, chairman* *Mr.J.A.C.Knops, secretary* *Mr.R.J.Oosterbaan, secretary*	

The discussion in each session was often conducted with a list of questions at hand, which list was prepared in advance by the Organizing Committee.

Conclusions and recommendations are presented in Section I of this publication.

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Section I

Synthesis of the discussions and contributions

1. DESIGN AND RESEARCH

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Contents

1.1 Introduction
Design and research problems

The design of a drainage system involves decisions on several inter-dependent elements:

a) the kind of system (e.g. surface, subsurface, mole drainage)

b) the kind of land shaping if surface drainage is chosen

c) the type of field drains (e.g. ditches, pipes, wells) if subsurface drainage is chosen

d) the lay-out of the system (field drains, collectors, outlets)

e) the depth and capacity of the drains.

The decisions must be based on certain criteria, which should formally be economic if the optimum system is to be obtained.

The Workshop was deeply concerned with the lack of precise knowledge on the various effects drainage may have on crops, soil, farm operations and environment. This lack of knowledge (primarily caused by the wide variety and complexity of the natural factors involved) renders it virtually impossible for the drainage engineer to develop very specific criteria.

Despite these problems,many drainage projects have been realized,especially during the last decades and seemingly with great success.The design of these projects is "more often than not still based on guides derived from experience rather than on analytical formulations"; as van Schilfgaarde (4.03) puts it. Apparently good decisions can be taken in large-scale and costly drainage schemes despite the lack of precise knowledge on their effects.

An explanation can perhaps be found when one realizes that the number of factors involved in the effect of drainage on agriculture is extremely large so that the effects of the different factors even out in a wide range of designs. Assuming one were able to express the beneficial effects, the harmful counter-effects and the costs as a function of a single variable representing different drainage designs, one would probably find that over a relatively wide range of designs the net benefit would not vary widely.

An example of a function offering a range of design options is depicted in Fig.1, which has been derived from Figs.6 and 7 presented by Eriksson (1.09).

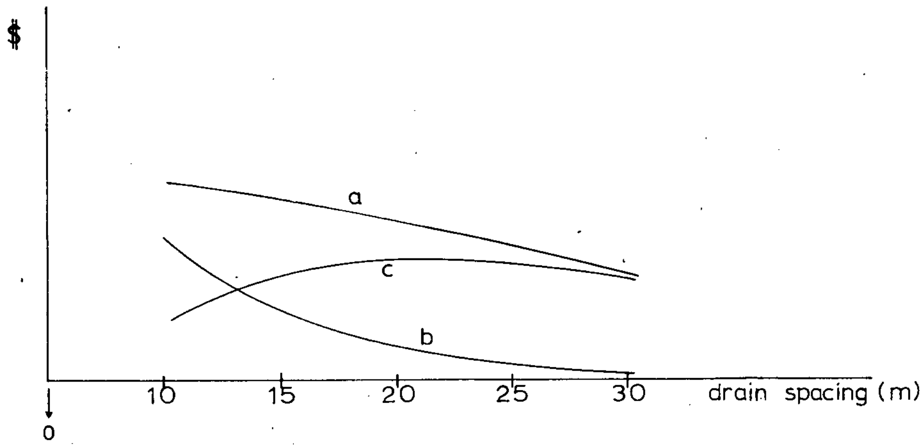

Fig. 1. Production (a), cost (b) and net benefit-functions (c) of drain spacing on a 60% clay soil in Sweden.

This figure reveals that the net benefit is practically independent of drain spacing within the range 15-30 m. The variables employed here are admittedly simple: the only design option considered was the spacing of subsurface drains; farm operational aspects, such as trafficability, were not included.

In line with the view developed above is the opinion of Dieleman (4.02): "The present status of knowledge and technology is considered generally adequate to meet short-term needs of drainage design and implementation in developing countries". Van Schilfgaarde (4.03) comments "I suspect that not much will be gained from further refinement of existing drainage theory", and "The challenge ahead is to imaginatively apply the existing catalogue of tricks to the development of design procedures". Zaslavsky (1.07) concludes: "The existing theoretical knowledge should be reduced to simple rules concerning design parameters".

The above statements do not deny that further research on drainage is required. The contrary is true, because there may be outright failure if one chooses a drainage design outside the safe range of options. In this context most participants of the Workshop pleaded for interdisciplinary research, studies in width rather than in depth. Dieleman (4.02), when referring to unsuccessful projects in developing countries, says that insufficient precise data are seldom the sole cause of the problem: "Rather it is unawareness of the effects that changes in the water regime, brought about by water development and irrigation, may have. It is also a lack of understanding of the broad inter-relationships between the various technical components as well as between these and non-technical factors". Van Schilfgaarde (4.03) calls for "a better definition of drainage criteria and expansion of the data base for crop response as well as trafficability. Closely related is the need to consider drainage problems as a part of a total water management scheme". Zaslavsky (1.07) supports the above opinions by saying "Much more good can be obtained today by adaptation of existing knowledge to proper practice than by new research. A different type of training of drainage engineers should be based on a good understanding of the phenomena involved and existing practices".

It is clear from the above quotations that the project engineer needs to have a good qualitative insight into which factors are really important in design and which are not.

Drainage projects should be realized on the basis of local or regional well tested experience. This implies a constant monitoring of the accomplished parts of the project and execution in stages. The stages should not only be area-wise; there should also be stages in the intensity and capital investment of the project. Zaslavsky (1.07) says that " First stage execution of a drainage scheme can serve as an experiment for a second stage. If, for example, the anticipated spacing between drains is 20-30 metres, one can first build drains at some wider spacing". In fact the whole history of The Netherlands presents an example of polder development in stages. This development is still continuing.

In summary it can be stated that an experimental approach, supported by a growing insight into the effects of drainage, still seems a better basis for the development of drainage than an analytical-mathematical approach by a few specialists.

Group 100 of the Workshop did not have the opportunity to discuss all the points listed at the beginning of this paper on which decisions must be made. In fact, under Point a (the kind of field drainage system to be adopted) the discussions centred around pipe and mole drainage. Point b was not treated, whereas Point c was partly treated in Group 100 and partly in Group 200. Under Point d (the lay-out of the drains) only the spacing of pipe drains received due attention; the majority of the papers submitted dealt with this subject. Drain spacing formulas, the determination of hydraulic conductivity of the soil, and the effects of drain depths or spacing on plant and soil conditions were extensively discussed (the last-mentioned subject under the heading Criteria). Point e was not discussed in Group 100 but was partly covered by Group 300.

1.2 Drains spacing formulas
Hydraulic conductivity

One of the tools available for the design of a drainage system is the drain spacing formula that relates system properties (spacing, depth, dimension etc.) to soil properties, hydrological conditions, and behaviour of the groundwater level. Many such formulas, valid for various idealized conditions, are available. When using these formulas, one faces several problems:

- natural conditions seldom lend themselves to idealization; the variability in time and in space is often so wide that it becomes difficult to find representative values;

- the groundwater level is not directly related to crop production, but indirectly, through a large number of physical and chemical soil responses to variations in groundwater level. Determining the effect of the groundwater level and the variations permitted therein is therefore very difficult.

Because of these difficulties the Workshop concluded that drain spacing formulas are only of relative importance in the sense that:

- they can give an order of magnitude of required drain depth and spacing;

- they offer the possibility of estimating the relative influence of certain conditions and their variations on required drain depth and spacing; they can thus be used to find out which conditions are important for drainage design and which are not (sensitivity analysis).

Refining the existing drain-spacing formulas generally makes little sense. For example several steady-state formulas are available for flow towards parallel drains in homogeneous isotropic soils and for drains not reaching an impermeable layer, e.g. the formulas of Hooghoudt, Dagan and Kirkham. All these formulas are based on the same principles and give quite similar results.

Only if new parameters can be introduced is the further development of drain spacing formulas useful. This holds, for example, for the entrance resistance (Wesseling, 2.10) soil anisotropy (Boumans, 1.03), and soil layering (Ernst, 1.02; Wolsack 1.01).

Concerning the question whether steady-state or unsteady-state formulas should be used for design, the Workshop concluded that both approaches have their merits and demerits. Steady-state formulas are simpler and therefore allow certain conditions to be introduced more easily, for example, flow above drain level, layered soils, soil anisotropy, and radial resistance. When it appears that watertable fluctuations (which always occur because drainage is an unsteady-state process) are determinant for crop production or soil conditions, the use of unsteady-state formulas has preference, provided that the determination of such parameters as drainable porespace or recharge poses no major problem.

If average watertable depths and discharges over prolonged periods are of primary concern, steady-state formulas are preferred.

One of the basic soil properties that has to be known when applying drain spacing formulas is the hydraulic conductivity. The Workshop agreed that because of the spatial and time variability of hydraulic conductivity it is very difficult to obtain reliable representative values. There was a strong feeling in the Workshop that the hydraulic conductivity should be determined from experimental drains rather than from point determinations. In principle one can experiment with a single drain, or a set of parallel drains, provided that there is a theoretical solution for the flow problem concerned.

To save on the number of measuring points and to get better representative values for the hydraulic conductivity, one can make use of the correlative method, by which the hydraulic conductivity is correlated with certain soil characteristics (Bailey, 1.12; van der Meer, 1.05).

1.3 Drainage criteria

A subsurface drainage system can be characterized by the two properties: depth and intensity. Intensity is defined as the ratio of drain discharge (in m/day) divided by the hydraulic head (in m) of groundwater midway between the drains. It is thus the reverse of the drainage resistance and its dimension is day^{-1}. This definition presumes a linear relation between discharge rate and hydraulic head.

The drainage criteria enable the depth and intensity of the drains to be chosen in such a way that the drainage aims are fulfilled. These aims are to obtain and preserve favourable conditions for plant growth and farm management by:

a) Avoiding too wet conditions during rainy periods;
b) Obtaining workable conditions shortly after rainy periods;
c) Controlling salinity in arid climates.

a) Avoiding too wet conditions

Too wet conditions have to be avoided because of damage to plants due to lack of aeration, deterioration of soil structure, loss of nitrogen by denitrification, and other reasons. Moreover, too wet conditions can cause excessive surface run-off, which mostly results in very wet situations in lower-lying areas. Sometimes, and in some places surface run-off can lead to soil erosion.

During rainy periods, moisture conditions in the top soil are strongly related to the depth of the groundwater table. The groundwater table depth is therefore a good measure of the effect of a drainage system during and shortly after rainfall.

Groundwater depth can be controlled by the depth and intensity of the drainage system. From the work of Wind and Buitendijk (1.33) it appears that drainage intensity is the most influential factor governing the groundwater table depths. Wesseling (1967) showed that the intensity of a drainage system as defined above lends itself very well to computing the depths of a fluctuating watertable under prevailing rainfall conditions. From a statistical analysis the frequency of exceedance of a certain watertable level can thus be expressed as a function of intensity and depth of the drainage system. The basic problem remaining is to find out the effect of watertable depth on crop yields.

b) Obtaining workable conditions

Workable conditions exist when moisture conditions in the soil are such that they do not impede farm traffic, cattle grazing, or seedbed preparation. All these operations require their own moisture condition, which may differ from one soil type to another. Most of the operations require a fairly dry topsoil with a soil moisture suction of 100 cm or more. The moisture condition of the topsoil in its relation to depth and intensity of the drainage system has been investigated by Wind and Buitendijk (1.11). They showed that intensity has only a slight effect on workability whereas the effect of drain depth is paramount. The reason for this is that tillage operations normally occur in fairly dry spells following rainy periods in which the groundwater table tends to equal drain depth regardless of drain intensity. The deeper the watertable, the drier the soil, and because its capillary conductivity is lower, the capillary rise is less so that the topsoil soon dries out by evaporation.

In certain types of soil, deep drainage can lead to over-drainage, causing drought damage in dry growing seasons. Drain depth has to be so chosen that the sum of damage due to insufficient workability and yield losses due to drought are minimal.

c) Salinity control is treated in the report of Group 400

In the past, drainage criteria were often based on avoiding too wet soil conditions. Intensity was therefore considered more important than depth. The feeling of the Workshop, however, is that nowadays because of intensified and mechanized farming, workability is at least as important. Drainage criteria must include both aspects.

The Workshop recommended that the research on drainage requirements be directed towards the use of integrated models with which the effects of drain depth and intensity on both crop yield and farm management can be studied. Because of the enormous variability of the weather in temperate climates, models should offer the possibility of studying the effects over a sequence of years. To incorporate the workability aspect, they should include the phenomena occurring in the unsaturated zone. Since computations of long-duration series with numerical models often turn out to be expensive, analogue models as the one proposed by Wind and Mazee (1.10) seem to offer better possibilities.

1.4 Drainage of fine textured slowly permeable soils

The hydraulic conductivity of slowly permeable clay soils normally decreases rapidly with depth. It is the structure of the topsoil, caused by biological activity such as roots and wormholes and human activity such as ploughing, that is mainly responsible for the possibility of water movement; in the subsoil the permeability is practically zero. Apart from the biological and human activities, swelling and shrinking due to wetting and drying can greatly determine the permeability. The problem that now arises is that the hydraulic conductivity not only changes with depth but also with time. The installation of a drainage system may, due to a change in the moisture regime of the soil, bring about a change in hydraulic conductivity.

Fine textured slowly permeable clay soils are often difficult to drain by means of a subsurface system, because even an expensive narrow spacing may not solve the problem. When the stability is sufficient, a system of mole drains combined with a pipe system can provide a satisfactory solution.

Subsoiling is also advocated as an efficient means of draining slowly permeable soils. Bailey (1.12) states that subsoiling often results in a kind of inferior mole drainage. Subsoiling shatters the soil and breaks it up to a certain critical depth, which with conventional equipment is usually not more than 40 cm. Below this depth, an inferior rectangular mole channel is created. When mole drains are being ploughed, the soil is also shattered to the critical depth; so the difference between deep subsoiling and mole drainage is not altogether very great.

Schuch (1.14) claims that liming and subsoiling to 80 cm are efficient contributions to soil improvement. With these measures tile drains can be widely spaced (up to 80 m). The soils described by Schuch are generally more silty than those described by Bailey. Furthermore, Schuch is clearly interested in quickly removing the excess water after intensive rains, but wishes to retain the normal rain water in the soil to cover evaporation needs in summer. Bailey, on the other hand, emphasizes watertable control.

With respect to the effectiveness of mole drainage, there are two important questions which are not yet fully understood:

- Firstly, it is generally assumed that the mole channel is formed at the expense of soil compaction around the mole channel. If so, how does the groundwater reach the mole channel? Is it by radial flow in accordance with the classical drainage concept, despite the compaction, or is it merely gravity flow through the major cracks that are formed by the shatter of the soil above the mole channels? Bailey (1.01) suggests that the subsurface flow concept is applicable. If so, what about the compaction? The Workshop agreed that, to get a better understanding of the functioning of these systems, research should be done on the flow towards mole drains.

- Secondly, it is as yet unknown what the lifetime of a mole channel in a clay soil is. Texture alone does not seem a sufficient indicator. There is a need for further research on soil properties (e.g. texture, structure, plasticity, lime content) that may determine the durability of mole drains. The research should be orientated towards a systematic classification of clay soils in relation to their suitability for mole drainage.

Slowly permeable clay soils generally have low infiltration rates. When high rainfall intensities occur, water is ponded on the soil surface and a surface drainage system is required. Little is known about the interactions between subsurface and surface drainage. De Jong (1.15) describes a situation in which a subsurface drainage system can eliminate the need for a surface drainage system, but this situation is probably not universal. The Workshop feels that more information on this subject is needed.

1.5 Other topics of interest

Group 100 was unable to discuss all drainage design and research needs exhaustively. The following points deserve more attention:

- Surveys: what aspects other than hydraulic conductivity and soil properties need to be systematically surveyed, and how should such a survey be set up?

- Layout: There are various alternatives for the lay-out of collector drains, which also depend on the design of the lateral drains. The discussion on length and slope of the laterals (are long laterals with zero slope advisable?) and length and type of collectors (are pipe collectors preferable to ditch collectors?) needs to be firmly re-opened. This subject has been partly treated in Group 200.

- Operation and maintenance: These aspects of drainage systems are often overlooked when the systems are being designed.

- Economic evaluation of drainage works: This subject has been dealt with rather superficially so far. (See also the conclusions of Group 400.)

- Transfer of research results into practice (reduction of complicated models to design tools): By means of sensitivity analysis it should be investigated what factors are important, and to what extent models can be simplified.

- Environmental aspects of drainage (salt, nitrogen, and phosphate pollution).

- Monitoring of existing drainage systems in farm land or in experimental fields; developing systems of data collection and data processing; instrumentations.

2. DRAINAGE MATERIALS

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Contents

2.1 Introduction

In the past 20 years field drainage in many countries has evolved from a purely manual job into a highly mechanized and automated operation. The techniques and skills developed have drastically reduced the labour requirements and have increased the speed and efficiency of installation many times over. Through this evolution it has been possible to keep installation costs down despite sharply rising costs of labour and materials.

The introduction of corrugated PVC and PE tubing in the mid-sixties and the application of laser-guided automatic grade-control systems had a considerable influence on the development of high-powered, high-speed trenching and trenchless drainage machines. At the same time, this evolution stimulated the development and use of cheaper and labour-saving drain envelope materials, which, in their turn, enhanced the further mechanization of drain installation and raised installation rates.

Apart from the technical improvements in applying granular envelope materials and the advent of factory-made pre-wrapped organic envelopes, there has been a rapid development in synthetic fibre fabrics.

The technological advances have been so rapid that drainage engineers have not been able to keep their place as trail-blazers for industry and contractors. New techniques and materials introduced have been followed by laboratory and field investigations on their performance instead of the reverse.

2.2 Developments in envelope materials

2.2.1 General

Ever since horizontal subsurface drainage has been applied, measures have been taken to prevent sedimentation in those soils where soil particles tend to migrate towards and into the drain pipes. The solutions have been many and varied, and depended on local conditions and the economical availability of large quantities of permeable porous materials.

By and large, the materials used can be classified as:

- granular materials, the most widely used of which are coarse sand/fine gravel;
- organic materials, a great many of which have been applied, the type mainly being determined by local availability and conditions; they include wood-chips, saw-dust, straw of different crops, corn cobs, peat litter, and recently coconut fibre;
- synthetic materials, which followed in the wake of plastic drainage pipes; the products may be woven or non-woven, bonded, needled or punched synthetic fibre fabrics of glass-fibre, nylon, polypropene, polyethene, or acryl.

2.2.2 Granular materials

Graded coarse sand/fine gravel from pit-run sources is still the most popular type of envelope for subsurface drains in semi-arid and arid irrigated areas. It is used in fine sandy, silty, and dispersive soils, and in unstable subsoils under high groundwater tables, where a voluminous, highly permeable drain envelope material is generally required (Winger, 4.04; Johnston, 4.02).

From the "filter rules" formulated by Terzaghi in 1921, civil engineers developed criteria for the gradation of reverse filters to protect earth fill dams from piping. In the USA both the Bureau of Reclamation and the Soil Conservation Service adapted these criteria for use in agricultural drainage. Each agency produced a set of specifications for graded gravel materials for different soil types (Winger, 4.04; Boers & van Someren, 4.09).

The differences between the two sets of specifications are quite small (Willardson, 2.02), although the Bureau of Reclamation, considering the envelope to be part of the complete drainage system, emphasizes the hydraulic function of the envelope (its permeability requirement), while the Soil Conservation Service emphasizes the filtration function.

Wherever coarse sand/fine gravel that complies with the specifications is locally available in sufficient quantities and at reasonable cost, it provides an effective and durable envelope. Its transport and handling costs, however, can be high, nor is it readily available everywhere.

These disadvantages, combined with the introduction of light-weight plastic drain tubing and the high installation rates obtainable with the new machinery, stimulated the search for a light-weight substitute in the form of synthetic fibre fabrics (Willardson, 2.02). Johnston (4.01), however, states that none of the synthetic fibre-fabrics available nowadays ensures the stability required when drains have to be installed in unstable subsoils with shallow groundwater tables.

In humid areas, well-structured stable top-soil brought directly onto the drain (often referred to as "blinding") performed the functions of an envelope. When drainage became more mechanized, more difficult soils (fine sandy and silty soils) were drained too, but it was found that "blinding" in such soils did not adequately protect the drains from sedimentation. It was thus realized that under certain soil conditions special envelope materials that completely surround the drains were required (Willardson, 2.02).

2.2.3 Organic materials

In the deltaic areas in Northern and Western Europe, natural granular materials (coarse sand/fine gravel, blast furnace products, lavalite) have hardly been considered for land drainage because of the high costs of transport and handling and the problem of heavy transport on the fields.

In these regions organic materials used as drain cover or envelope have performed quite satisfactorily. The materials include peat-moss, fibrous peat, straw, reed, heather, brushwood, and woodchippings. Of these, fibrous peat has proved the best in The Netherlands and is still widely applied.

Until and shortly after the introduction of mechanized installation techniques, the materials were applied loosely, mostly covering only the top 3/4 of the drains. Drain line failures were often caused by sediment inflow from below and were not related to the use of ineffective materials. The introduction of corrugated plastic tubing, the steep rise in labour costs, and the further mechanization and automation of drainage works made it necessary to develop and process cheaper and more time saving envelope materials. In most West-European countries the stage has now been reached that whenever an evelope is required, pre-wrapped pipes are utilized. The most commonly used materials are: coconut fibre, peat litter, a mixture of peat and coconut fibre, and flax straw. These factory-made pre-wrappings should have a minimum envelope thickness of 7 mm (Knops et al., 2.15).

In the humid eastern parts of the USA and Canada, the organic cover and envelope materials never found the same wide application as in Europe. The reasons are probably their unknown life expectancy, the difficulty in handling the usually bulky material, and the fear that the organic matter might undergo chemical and biological reactions in the soil, resulting in clogging problems (Willardson, 2.02). Moreover, organic materials, being compressible, do not provide sufficient lateral support for PE-pipes, which may thus become excessively deflected, eventually causing pipe failure. For the more rigid PVC-pipes, however, this constitutes a lesser problem.

2.2.4 Synthetic materials

Drainage programmes are increasing in the humid and arid regions of the world, and in both the developed and the developing countries. FAO estimates that, to control waterlogging and salinity in the irrigated areas of developing countries, 52 million hectares of land need to be drained in the next 15 years (Dieleman, 4.02). A great many of the soils will fall within the range of difficult ones (fine sands, silts, and dispersive soils), in which drain envelopes will be required to protect the drainage system against sedimentation and to improve the flow of water in the area immediately surrounding the drains.

The high costs and scarce supply of suitable granular or organic materials, and in many areas their absolute non-existance, will make it necessary to look for low-cost substitutes, either in the form of manufactured synthetic fibre fabrics or the use of soil stabilizers.

In the traditional drainage countries of Western Europe, drainage is being extended more and more into lighter textured soils, while on the other hand drainage works are being executed under more adverse conditions. Both factors cause temporary instability problems.

These two developments have led to an increased need for drain envelopes. As good-quality organic fibrous materials are becoming scarce, there is an increased tendency to accept synthetic envelope materials, preference being given to the more voluminous type for fine light textured soils (Knops et al., 2.15).

In the USA and Canada, however, there is evidence that the commercially available woven and non-woven "thin" fibre fabrics perform well in light textured soils (Irwin and Hore, 2.01; Willardson, 2.02). This difference in preference is related to the fact that traditionally in Western Europe much smaller pipe diameters were used than in North America (50 mm against 100 mm).
Because of the rapidly increasing demand for drain envelope materials, there is an immediate need to learn more about the most efficient and effective use of synthetic fibre fabrics as drain envelopes.

2.2.5 Soil conditioners

It has long been recognized that the structure of a soil can be improved by adding suitable conditioners. Two types of conditioners, with distinctly different stabilizing actions, have been developed:

- the polymer solutions, such as polyacrylamide, polyurethane, and lignosulfonate;
- the polymer emulsions, such as polyvinyl acetate, asphalt, and rubber emulsions.

These conditioners could be applied to create highly aggregated drain envelopes. In slowly permeable, heavy soils, they could improve the trench backfill permeability.

The provisional results obtained by Dierickx and Goossens (2.04) and Zaslavsky (2.07) show promising perspectives for their use in land drainage. Even when very low concentrations were applied to both cohesive and non-cohesive soils in permeameter models, high permeabilities were obtained and sustained under prolonged tests (Dierickx and Goossens, 2.04). For fine-textured soils, the polymer solutions offer the best prospects, as no drying is required to obtain optimal stabilizing action. These soils treated with polymer emulsions need some drying after treatment. Swelling and shrinking clay soils should not be air-dried because aggregates may desintegrate by explosion upon wetting. Coarse-textured soils treated with either type of conditioner need air-drying to ensure optimum stabilization action.
Another practical complication is that to obtain optimum results there must be an optimum soil moisture content at the time of treatment. Howfar, this problem can be overcome with solution-type conditioners needs further investigation.

The initial results and cost estimates show promising perspectives for the use of solution-type conditioners in cohesive soils, but further research into what conditions influence its functioning is required. This includes such aspects as:

- the longevity of the stabilizing action in relation to soil water, soil chemicals, and microbiological activities;
- the effects of soil loads on the stability of the aggregates;
- the influence of swelling and shrinking on the aggregates.

Research is also required to develop a suitable and economical technology for the application of the soil conditioners.

2.3 Envelope requirements

2.3.1 Functional requirements

The two functional requirements of a drain envelope are:

a) Selective filtration function: the material should prevent the entry of soil particles that would otherwise silt up and clog the drains, block the perforations, and block or clog the envelope.

b) Hydraulic function: the material should maintain or create a highly permeable zone around the drain, thereby improving the rate of water entry.

A third function often referred to is the stabilizing function, i.e. the material should provide suitable bedding for the drain and stabilize the soil material of the trench invert.

The first two requirements are contradictory in their demands on the composition of the envelope. The groundwater flow converges towards the drain and concentrates further towards the entrance openings in the drains. This entails a sharp increase in the hydraulic gradient in the close vicinity of the drain. As a consequence, the risk of soil particle movement towards and into the drain is intensified. When a highly permeable zone around the drain is introduced, the number of pore-connections on the boundary between soil and envelope will increase, thereby decreasing the hydraulic gradient and reducing the risk of soil particle movement. The parameters of paramount importance for drain envelopes are therefore their total porosity, pore size distribution, permeability, and thickness.

The permeability, however, will be influenced by the soil particle movement towards and into the envelope (blocking and clogging). In the soil abutting the drain envelope, a natural filter may develop as the fine colloïdal clay and fine silt particles are removed by the flow, the larger particles and soil aggregates "bridging" the pores of the envelope, thus stabilizing and restraining the soil (Reeve, Zaslavsky).

As these processes are not yet well quantified, research should be conducted to study the inter-active processes between the parent soil material and the envelope and their effects on envelope permeability.

2.3.2 Standard requirements

In view of the rapidly increasing use of synthetic fibre fabrics as drain envelopes, there is an urgent need to develop standards for their mechanical and chemical composition. Properties to be considered are:

- tensile strength; the materials should be strong enough to bridge the corrugations without tearing or sagging into the valleys of the pipes and to withstand stresses during installation (Willardson, 2.02; Reeve);
- tear and puncture resistance; the material should withstand the stresses during construction, transportation, and installation;
- abrasion resistance; the material should be strong enough to avoid wear and tear during transportation and installation;
- ultraviolet light; the quality of the material should not deteriorate during storage;
- chemical resistance; the material should be inert to any soil chemical;
- micro-biological resistance;
- swelling properties; pores in the material should not be influenced negatively through swelling.

Existing test procedures and equipment have to be adapted or new ones developed. Acceptability levels for the physical strength requirement of the fabric need to be defined (Ochs, 2.06).

Research must be conducted to determine those soil chemicals (acids, bases, organic acids) which may be aggressive to certain synthetic materials.

2.4 Need for envelopes

Whether an envelope is required, and if so, what functional requirement will be decisive for its composition depends on the soil type (texture and structure), the "wetness" condition during installation, and whether precipitation of complex iron, manganese or magnesium compounds is to be expected. There are, however, no established criteria for determining when a drain envelope is required (Willardson, 2.02). The drainage engineer generally bases his decision on the use of envelopes and on the knowledge acquired from experience from laboratory and field experiments, and from evaluatory field research.

Guidelines have been compiled on the need for envelopes and the applicability of currently available drainage materials (Knops et al., 2.15). These, however, are only valid for the regional conditions for which they were compiled and cannot be transferred to other areas with different soils and drainage conditions.

In the past, several attempts have been made to determine characteristic values for particular soil textural and structural parameters so as to classify a soil's sensitivity to internal erosion. The uniformity coefficient (D_{60} over D_{10}) was first introduced to characterize soil stability. Sherard (1953) elaborated this by adding a measure for the aggregate stability: the plasticity index. Olbertz (1965) supplemented this further with the clay-over-clay-plus-silt ratio. Zaslavsky (2.07) proposed the use of the wet-sieving analysis, and Feichtinger and Leder (2.05) introduced a simple qualification test for cohesive soils. The problem, however, is that the results of these methods are greatly dependent on local conditions such as soil type, type of clay minerals, lime content, organic matter, salinity, and alkalinity, thereby prohibiting their general application (Kuntze, Willardson). Willardson (2.02) proposed the use of the concept of a critical hydraulic failure gradient. This is the gradient at which the soil loses its structural stability so that "quick" conditions occur. Zaslavsky (2.07) quotes work by Zaslavsky and Kassif (1965) where nearly the same principle has been described for erosion phenomena.

In cohesive soils, the gradient at which structural failure occurs, thus causing the progressive movement of soil particles, is many times larger than the gradient in granular soils.

Whether or not this gradient will be reached or exceeded depends on the soil type and the drainage conditions (depth, spacing and diameter, presence of envelopes, discharge/permeability ratio) (Willardson).

For arid-region soils in the western part of the USA, it appears to be related to the D60-size of the soil (Willardson).

To deepen our insight into the soils' sensitivity to internal erosion and the processes influencing soil particle movement, it is considered that research along the lines proposed by Willardson should be conducted. This research will provide us with the necessary information to develop a reliable methodology for predicting the need for an envelope in any soil type and for any drainage condition (depth, spacing, diameter).

2.5 Research of envelope materials

2.5.1 Introduction

In the past twenty years, rapid technological developments have taken place in drainage techniques and materials. The manufacturers of drain envelope materials had little or no knowledge of the required properties and were merely interested in finding new markets for their products or in increasing their turnover. This continuous supply of new materials placed a heavy burden on the drainage engineer and his supporting research institutions as they had to continuously judge the practical applicability of these new products.

The evaluation type of research needed for this purpose usually consists of the following steps:

- Based on his experience, the engineer makes a first selection through visual inspection;
- Potentially suitable materials are then subjected to laboratory tests, either in permeameter models or in sand tanks whereby the groundwater flow in the immediate vicinity of a drain is simulated as closely as possible. (See papers 2.01, 2.06, 2.10 and 2.11). These tests are usually performed in homogeneous fine sands to obtain consistent and reproducible results. The results are then compared with those obtained with products of known performance under field conditions. The information thus obtained, however, is not adequate to predict with complete confidence the performance of materials under actual field conditions;
- The final testing to decide the practical suitability of new drain envelopes is done in experimental fields or through evaluatory field research.

The disadvantages of these testing procedures are:

- the long time (three to five years) between the introduction of a new material and the "green light" for its practical application;
- the extremely high recurrent expenditures;

- the performance cannot be predicted for conditions other than those tested.

The time-lag is becoming unacceptable to users and producers alike.

For the development of improved drain envelope materials, it is of great importance that research be conducted to investigate and quantify those parameters which are crucial for an envelope's performance under all conditions. For this purpose design criteria should be developed for the porosity, pore-size distribution, and permeability of envelope materials for different soil types.

2.5.2 Research approach

The total porosity and pore-size distribution are decisive for the selective filtering and the hydraulic properties of drain envelopes (Knops, 2.10; Willardson, 2.02). The total porosity should be equal to or greater than that of the drain abutting soil to guarantee the maximum number of pore-connections at the boundary between soil and envelope and thus prevent the convergence of flow.

The permeability of the envelope, which is related to porosity and pore-size distribution, should be several times that of the surrounding soil to decrease the hydraulic exit gradients and so diminish the risk of soil particle movement (Willardson, 2.02; Cavelaars; Knops et al., 2.15).

Pore-size distribution

Three methods are used to determine the sizes of openings and their distribution in synthetic and organic fibre fabrics.

1. Dry-sieving method; the fabric is fixed in a standard sieving frame and acts as a sieve. The pore-size distribution is determined by standard sieving with either closely graded sand-fractions (Ogink, 1975) or glass-beads (Ochs, 2.06) whose limits correspond to those of standard sieve diameters.
2. Optical method; the number and area of the voids or openings in the fibre fabric are measured by an image-analyzing computer (Irwin and Hore, 2.01).
3. Suction method; the pore-size distribution is derived from the moisture retention curve of the fabric, postulating that the water is held in the pores by capillary force only.

The dry-sieving and optical method are well suited to determine the pore-size distribution of thin woven fabrics with a regular pore-size pattern such as woven nylon "socks". The optical method is effective for thin, non-woven, spun-bonded fibre fabrics such as glass-fibre (Irwin and Hore, 2.01). The suction method is preferred for the thicker organic and synthetic fibre fabrics (Knops, 2.10).

An advantage of this method is that with thicker materials, which are usually compressible under load, the method can account for the influence of a soil pressure and the spatial variation in degree of compression due to the corrugations and valleys in the pipes on the pore-size distribution.

More research is needed to study the reliability and reproducibility of the results obtained with the different methods.

Hydraulic conductivity

The hydraulic conductivity of an envelope should be (and should remain) several times that of the drain-abutting soil so as to ensure the lowest possible resistance for the flow towards and into the pipe (Dierickx, 2.03; Cavelaars). A high hydraulic conductivity will decrease the risk of high hydraulic exit gradients and consequently reduce the risk of soil particle movement towards and into the drain (Dierickx, 2.03, Willardson, 2.02). Small soil particles tend to move under the influence of the groundwater flow and may block or clog the drain envelope. Whether an envelope will become less permeable depends on the pore-sizes of the envelope, the thickness of the envelope, the particle sizes of the soil, and the hydraulic gradients the soil can sustain. It is therefore necessary to study the blocking and clogging processes and their influence on the permeability of the drain envelope, in combination with studies on the hydraulic gradients a soil can sustain.

The approach outlined by Knops (2.10) should lead to the development of design criteria that will determine the composition, thickness, and pore-size distribution of synthetic or organic fibre fabrics. These reference values and eventual specifications (Section 3.2) should then offer manufacturers and users the instruments to dimension fibre fabrics for drain envelopes to match the soils to be drained.

Experimental fields

The envelopes thus developed should be tested under actual field conditions before any large-scale application can be considered. The set-up of experimental fields should be such that the results obtained are reliable and transferable. The performance of the drain envelopes should not be judged only on the basis of drain discharge measurement, but also on the head losses required for the flow towards and into the drain.

Guidelines on how to set up experimental fields for single drain line testing and what measurements and observations ought to be carried out were published by Dieleman and Trafford (1976) and were not a topic of discussion at the Workshop.

2.6 Hydrological properties of drainage materials

2.6.1 Drain diameter

Most of the drainage formulas (Hooghoudt, 1940; Kirkham, 1958) used for design purposes are based on the assumption of completely permeable drains (so-called "ideal" drains) flowing full without any back-pressure.

With the flow converging radially towards the drain, the hydraulic gradient will increase towards the drains and consequently the flow velocity will increase (Dierickx, 2.03; Willardson, 2.02).

The larger the drain diameter, the smaller the convergence, and the lower the hydraulic gradient. It is both impractical and uneconomical to manipulate the drain diameter to keep the hydraulic gradients in the immediate drain vicinity small enough to avoid soil particle movement. Moreover, actual drains, with their relatively small area of inflow openings, are far from "ideal". Because of the additional concentration of flow lines towards the inlet openings, there will be a considerable extra flow resistance as compared with the flow to an "ideal" drain of equal diameter. This extra resistance was defined by Engelund (1957) as "entrance" resistance. In theory, it can be accounted for by introducing the concept of "effective" radius. Thus a real drain can be replaced by an "ideal" drain with a smaller radius causing the same total flow resistance (Nieuwenhuis and Wesseling, 2.09).

Most of the design formulas assume that no water is standing above the drain. For so-called "optimum" drainage conditions (drain just running full and "ideal"), a certain minimum drain diameter is required. This diameter can be derived from theories that apply the hodograph method (Wesseling, 2.08). Childs and Youngs (1958) pointed out that under non-optimum conditions, with too small a drain diameter, water will stand above the drains and the water table midway between the drains will not rise in the same proportion. This corresponds with the fact that submerged drains are more effective, as has been proved by Wesseling (1964).

In practice it is assumed that the minimum drain diameter is determined by its hydraulic properties and transport capacity, rather than by the drainage conditions, because of the complexities involved such as complicated theories, non-ideal drains, use or non-use of envelope materials, and permeability (Wesseling and van Someren, 1972).

2.6.2 Perforations and envelopes

An increase in the density of the perforations in the pipes is the most effective way of decreasing the entrance resistance and approaching the "ideal" drain (Nieuwenhuis and Wesseling, 2.09).

Research findings by Dierickx and Willardson indicate that the nowadays applied distribution of perforations in corrugated plastic tubing ought to be reconsidered, especially in the light of further decreasing the risk of occurrence of hydraulic failure gradients in the vicinity of the perforations. Willardson (2.02) indicates a significant reduction in hydraulic conductivity at large gradients. Zaslavsky (2.07) gives an approximate formula for the critical gradient.

For a specific opening area per running metre of pipe, a large number of small perforations, evenly distributed, is more effective in decreasing the entrance resistance than a small number of larger perforations.

The number and distribution of perforations is less important when the drain is provided with a permeable envelope. Even "thin" permeable envelopes, with sufficient tensile strength to prevent sagging into the valleys, will decrease the entrance resistance considerably and prevent the occurrence of high hydraulic exit gradients, provided that each valley of the corrugated tubing is supplied with at least one perforation. The disadvantage of "thin" envelopes, however, is that they are apt to block.

Experiments with different types of envelope material showed that a decrease in resistance could be obtained more easily by increasing the envelope thickness than by larger pipe diameters or more perforations (Wesseling and Homma, 1967a). In view of both the hydrological properties and the critical hydraulic gradient of failure it is better to use a smaller diameter drain plus permeable envelope than a larger drain without an envelope.

Practical experience (Knops et al., 2.15), supported by theoretical analysis (Nieuwenhuis and Wesseling, 2.09), proves that an envelope thickness of 5 to 10 mm with a permeability at least 10 times that of the surrounding soil will decrease the entrance resistance sufficiently.

Increasing the thickness any further will have no effect on the entrance resistance, but decreases the radial resistance and consequently increases the 'effective' radius.

High permeability values of the envelopes are less effective in reducing the entrance resistance.

2.6.3 Clogging or blocking of envelopes

A decrease in the permeability of a drain envelope by soil particles blocking the pores on its surface ("thin"-envelopes) or entrapped inside it ("thick"-envelopes) will increase the entrance resistance (Nieuwenhuis and Wesseling, 2.09) and will result in higher hydraulic exit gradients. Depending on the envelope thickness, the original permeability and the rate of its decrease, clogging or blocking can lead to a rapid failure of the drainage system.

If the permeability of the clogged part of envelopes thinner than 10 mm has decreased to a lower value of that of the abutting soil, there is a pronounced increase in entrance resistance and in radial resistance resulting in a smaller effective radius. If the envelope is thicker than 10 mm there is only a slight increase (Nieuwenhuis and Wesseling, 2.09).

In both cases the possibility of water rising above the drain increases, but as this rise is less than 0.10–0.15 m it will have a negligible effect on drainage performance (Wesseling, 2.08).

2.7 Hydraulic design

2.7.1 Transport capacity

The capacity of the drainage system should be such that the pursued objective of optimum drainage will be met without exceptionally high pressure losses due to the flow of water through the pipes.

Laboratory investigations on the transport capacities of pipe materials applied in land drainage showed that for well-laid clay tiles and smooth rigid PVC and PE pipes, the relation between friction factor (f) and Reynolds number (R_e) can be given as:

$$f = a\, R_e^{-0.25}$$

where

a is a constant representing the intercept of the straight line relationship when log(f) is plotted against log(R_e).

This results in the flow formula:

$$v = C\, R^{0.7}\, S^{0.57}$$

with

- v = flow velocity
- C = constant
- R = hydraulic radius
- S = hydraulic gradient

For the constant a, values of 0.4 valid for well-laid clay tiles and smooth rigid PVC-pipes are reported (Wesseling and Homma, 1967).

For corrugated tubing, the well-known Manning formula can be applied:

$$v = 1/n\ R^{2/3}\ S^{1/2} = k_m\ R^{2/3}\ S^{1/2}$$

For corrugated pipes, values of $k_m = 70$ or $n = 0.014$ are reported (Wesseling and Homma, 1967; Hermsmeier and Willardson, 1970).

Studies on transport capacities under field conditions resulted in lower values than found in the laboratory (Wesseling and Homma, 1967b; Trafford et al., 1972; Dekker, 2.13). For practical design purposes, therefore, a reduction of 20-30 per cent in the transport capacity obtained in laboratory tests is often applied.

In determining the transport capacity of a drainage line two principles can be followed:

a) the transportation principle which implies a full-flowing pipe over its entire length at a constant discharge (uniform flow);
b) the drainage principle with a constant inflow per unit drain length and a gradually increasing discharge (non-uniform flow).

The transportation principle, which gives a constant hydraulic gradient over the entire length of line considered, is generally applied in the USA and the Federal Republic of Germany. The hydraulic grade line at design discharge is not allowed to rise above the drain (just flowing full without over-pressure), which entails that the design hydraulic gradient equals the design slope of the line.

The drainage principle gives a hydraulic gradient that increases from zero at the top end to a maximum, equal to the one for uniform flow, at the outlet end. The design gradient is taken as the average gradient over the whole length of the drain line.

In comparing the two principles, Wesseling and van Someren (1972) and Cavelaars (1974) concluded that:

a) with an equal total outflow, the average gradient for non-uniform flow is only approximately 1/3 the gradient for uniform flow;
b) with equal gradients, the discharge of uniform flow is approximately 0.6 times that of non-uniform flow.

Concluding, it can be stated that when the drainage principle is applied either smaller diameter pipes will suffice or the maximum allowable area to be drained by a drain of a given diameter is increased by about 20 per cent.

2.7.2 Hydraulic grade lines and slope of drain line

The use of the drainage principle will cause the hydraulic grade line to rise above the drainage line and will automatically establish a surcharge in the system at design discharge, especially when steeper slopes are applied (Cavelaars, 2.12).

For the drainage performance, the drain line does not need a slope at all. When the slope is less than the hydraulic grade, the drain will be submerged. This is beneficial for the approach flow conditions, because the exit gradient will be less (Section 2.6.1).

In flat areas it is common to give drainage lines a minimum slope of 0.05-0.2 per cent, but the design slope should never be steeper than the average gradient given by the design based on the drainage principle. This implies that the pipes are full-flowing over their entire length and are thus used to their full capacity.

In steeply sloping areas, applying the drainage principle could result in extremely high surcharges in the system. Trafford suggested applying a design slope equal to the average hydraulic grade over the first third part of the drain-line (see Wesseling and van Someren, 1972). For the design of mole channels no surcharge can be allowed because of stability reasons; hence their design should be based on the transportation principle.

Neither approach agrees with reality. The drainage principle comes closest to it, but because of the possible surcharge in the drainline the inflow per unit length of drain will not be constant. Owing to this phenomenon, difficulties arise in the interpretation of measured hydraulic gradients in the field (van der Beken et al., 1972; Trafford, 1973). In this respect the question arises whether a two-dimensional approach to a drainage system is allowed (Wesseling).

2.7.3 Lay-out of drainage systems

The following drainage lay-out systems can be distinguished:

a) Singular systems; lateral lines of rather short-length (100-250 m) single-diameter pipes, flowing into open collector systems.

b) Extended singular systems; lateral lines of extended length, discharging into open collector systems. These systems offer an attractive alternative in large-scale drainage with wide spacings (Boumans, 4.07). In this system the laterals are bound to be built up of sections with increasing diameters. If the design is based on the drainage principle, each diameter can only be used to a certain percentage of its maximum length (drainable area) since otherwise the hydraulic gradient would become considerably larger than the average slope of the line (Cavelaars, 2.12).

c) Composed systems; lateral lines flowing into buried collector lines. These systems are commonly found in areas where sufficient surface slopes are available, closed collectors needing a steeper gradient than open collectors because they require a large transport capacity. These systems also predominate in irrigated areas (Winger, 4.04), where they save on losses of fertile land (Cavelaars, 1974).

   In the near future these systems may also become economically attractive in flat humid areas because the technological advances being made in lateral and collector connecting materials and installation techniques will soon enable a totally mechanized installation. In addition, the developments in maintenance equipment are making it possible to cut down on the number of structures required for access purposes.

2.8 Drain installation

2.8.1 Vertical alignment

Drain lines should be installed at their design slope and depth. But, despite the automatic laser-guided grade-control systems applied nowadays, deviations from the design slope are quite common. Undulations larger than the pipe diameter are unacceptable as this would cause air entrapment, resulting in pressure build-up and malfunctioning of the system, the results being comparable to silt blockages. These inaccuracies have been observed more frequently in drains installed by trenchless techniques than in those installed by trencher. This can probably be attributed to the much higher installation speeds obtained with the trenchless techniques. Whether it is also related to the mechanisms governing the depth regulation has not yet been established. There is no clear proof because there is as yet no proper technique available to monitor the position of a drain line installed by the trenchless method. Thus, setting specifications for acceptable deviations from the design slope has at present no practical meaning.

The suggestion that air entrapment caused by undulations in the line can be avoided by providing a maximum allowable slope must be considered incorrect, as can easily be shown with a simple calculation: the 0.2 per cent slope normally applied in flat areas means a difference in height of only 2 mm over a metre.

2.8.2 Installation conditions

Installation during wet weather and/or when groundwater levels are high can impair the functioning of the drainage system by structural deterioration, or even lead to the complete failure of the system. Depending on the soil type, the trench backfill may become far less permeable than the undisturbed soil; unstable backfill materials may place extreme demands upon the envelope material, while sudden high hydraulic gradients will occur because of high groundwater tables (Knops et al., 2.15; Willardson, 2.02).

Field research into the causes of drain system failure has revealed that failures could very often be related to installation under "wet" conditions (Knops et al., 2.15; van Someren, 1965; Cavelaars, 1966). Especially for soils sensitive to structural deterioration, it pays off to install drainage systems under dry conditions, as the merits will easily compensate for the demerits of damaging standing crops (Carter et al., 1974).

2.9 Maintenance

In all types of water-conducting systems, maintenance is a prerequisite for lasting performance. In subsurface drainage systems there are various causes that may impair their proper functioning. These can be: broken or strongly deflected pipe sections, clogging and blocking of the line by soil sediments, chemical compounds, and plant roots.

The sedimentation danger is especially hazardous in sandy and silty soils, and in soils susceptible to structural deterioration when drains are installed under "wet" conditions. Extreme sedimentation may occur immediately after installation when the trench backfill has not yet consolidated. Well-designed envelope materials and proper installation practices can diminish such hazards. Chemical precipitates like iron, iron/sulphur, and manganese compounds may clog drains and envelopes alike. Plant roots penetrating through pipe joints or perforations may fill up the pipes over extended lengths. Busser and Scholten (2.14) describe the various ways of overcoming these problems.

These authors also analyse the impact of partial clogging on the transport capacity of the system, and advise that it be taken into account in the design of the system. A 25 per cent decrease in effective cross-sectional area will result in a transport capacity decrease of 35 per cent, or in a doubling of the required hydraulic head.

The technique of high-pressure jet cleaning (80-100 atm. at the pump) is not advised in The Netherlands because a "quicksand" condition may develop around the drain, causing the immediate entry of large amounts of sediments. This technique should not be applied in drainage lines that have no envelope since the piercing action of the water jets through the perforations will destroy the natural filter, build-up around the perforations.

In areas where drainlines are installed with a well-graded sand/gravel envelope, high-pressure jet cleaning can be applied with a lesser risk provided that the contractor is well aware of the possible dangers involved and keeps the jet moving continuously.

In contradiction to the opinion of Cavelaars and Busser, Willardson and Winger believe that sands can be removed from the line provided that the pressure is kept up when the hose is being withdrawn from the line. In this respect it was concluded that more research is needed to study the possibilities of removing sediments from drain lines by high and medium pressure jet cleaning.

Also the effects of high-pressure jet cleaning on envelope materials should be investigated more closely.

Iron, manganese and sulphur compounds clogging drainlines and perforations can easily be removed by medium-to-low (20 atm.) pressure jet cleaning, provided that the deposits have not been too long subject to drying-out and cementation.

In wet periods and with sufficiently high discharges from the drainage system, even dry-rodding will enhance the performance of the system because the loosened iron deposits will be carried away by the water. Whether ochre precipitations inside the envelope material can be tackled by jet-cleaning is still not known and should also be subjected to further research.

2.10 Chemical and biological clogging

One of the most familiar drain-clogging problems is the formation of ochre. The chemicals usually responsible for clogging drain pipes, perforations, joints, and envelopes are iron compounds, sometimes iron-sulphur compounds or manganese compounds (Grass, 1969). In principle, ochre formation arises from the dissolved Fe(II) which oxidizes to Fe(III), leading to the precipitation of insoluble compounds. The ochre formation can be induced by chemical or biochemical oxidation. The chemically precipitated Fe, under water, is more porous and less a clogging agent than the biochemical precipitation (Ford, 2.16).

The biochemical ochre formation finds its origin in numerous autotrophous and heterotrophous iron-organisms (Kuntze, 2.17), and is the primary cause of drainline clogging (Ford, 2.16). The composition of the deposits may vary, depending on the circumstances under which they are formed (Kuntze, 1966). In soils rich in organic matter, sulphur plays an important role (Ford and Calvert, 1969; Ford and Beville, 1970). Complexing agents can also contribute to iron-ochre formation. In this respect it needs to be established whether citric acid, humic acid, tannins, lignins, or other complexing agents that can form Fe or Mn complexed colloidal particles capable of clogging drains and increasing entrance resistance in the zone abutting the drain envelope occur naturally (Ford, 2.16).

At present the clogging phenomenon cannot be prevented, nor is there any envelope material capable of solving or reducing the problem.

The origin of the problem, as described by Kuntze, can be allochthonic or autochthonic. The allochthonic problem is a permanent, continuous problem, because of its alien origin; the autochthonic problem is a temporary one, because of its indigeneous origin. To control the autochthonic problem successfully, a combination of measures are suggested by Kuntze (2.17). If the problem is of a short duration (2-4 years), the 'antoc'-envelope may be a possible solution (Kuntze and Scheffer, 1974; Scheffer and Kuntze, 1975). There are no effective and economical measures of controlling or preventing the allochthonic problem.

From a practical point of view, it is vital that ways be found of determining beforehand whether an ochre-formation problem is likely to arise and, if so, what its origin will be. In addition, the investigations leading to effective control measures should be continued.

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3. INSTALLATION METHODS

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Contents

3.1 Trenchless drainage method

3.1.1 Introduction

The plough-in method of drain installation has been termed "trenchless", in contrast to the "trenching" method which involves both soil excavation and back-fill operations. The trenchless method places the tubing at a prescribed depth in an open space beneath a temporarily displaced column of soil. The plough blade is designed to lift and split this soil column as it moves forward. The lifting action causes a deformation and a disruption of the soil upwards and towards both sides. The tubing is fed in behind the plough blade where the soil falls back around the pipe.

The two major questions associated with the trenchless technique, as indeed with other methods too, are

(1) can the tubing be laid on grade at the desired depth within acceptable limits, and

(2) does the condition of the soil after installation allow unrestricted flow of water into the drain? (Reeve 3.09).

3.1.2 Soil disturbance with narrow tines

The basic shape of the plough blade used for drainage operations is an inclined narrow tine. Two types of soil disturbance can occur with narrow tines. At shallow working depths the soil is displaced forwards, sideways, and upwards throughout the whole working depth. This type of disturbance causes fissuring and loosening and is termed crescent failure.

At greater working depths the tine causes crescent failure in the upper soil layers, but in deeper layers the soil moves forwards and sideways only (lateral failure) and this causes deformation (movement with no change in density) or compaction. The depth at which the transition from crescent to lateral failure occurs is termed the critical depth (Spoor 3.08). Below the critical depth, where only deformation occurs, the soil movement can be described as a sequence of steady state situations of soil flow. The streamlines coincide with the sliding lines as have been described by Prandtl (1921). The flux of soil during a steady-state situation is chosen in some arbitrary way. The deformation of the soil related to the friction angle and the sideward distance to the plough blade can be derived from the known streamlines and assumed displacement (Boels 3.11). The leading section of the tine at working depth controls the type of soil disturbance. The tine aspect ratio (working depth divided by tine width) and tine inclination to the horizontal in the direction of movement (rake angle) are implement factors that influence the critical depth. The smaller the aspect ratio and rake angle, the deeper the critical depth. Increasing moisture content tends to decrease the critical depth. A hard topsoil on a soft subsoil means a shallower critical depth than in a uniform soil. Pure deformation is most likely to occur only under wet soil conditions. In the zone with crescent failure the soil will be loosened without deformation. On soils with high clay content, smearing is observed on the soil-implement interface (Winger 4.04). No data are available on the long-term influence of smearing on the hydraulic conductivity.

3.1.3 Hydraulic conductivity related to soil disturbance

As pointed out above, three types of soil disturbance can be distinguished: loosening, compaction, and deformation without compaction or loosening. Loosening and compaction change the pore-size distribution and will influence the original hydraulic conductivity of the soil. Loosening increases conductivity; compaction reduces it. These relations can easily be determined.

If soils have very few large pores per unit area, soil deformation can result in the blocking of these pores, thereby reducing the conductivity. On sandy soils where pore distribution largely depends upon the particle size distribution, deformation does not change the initial hydraulic conductivity. The

TABLE 1. Ratio between the hydraulic conductivity in vicinity of the drain pipe with trenchless installation (K_2) and trenching installation (K_1) based on measurements of drain discharge and height of groundwater table

Soil type	Drain depth (m below surface)	Country	$\frac{K_2}{K_1}$	Remarks	Reference
Sand	1.1	Neth.	1.0	average 1975-1977 (14 pipes)	Naarding
Sandy loam	1.2	Denmark	0.5	average 1972-1978 (6 pipes)	Olesen
Sandy loam	1.2	Denmark	0.6	average 1972-1978 (10 pipes)	Olesen
Loam	1.2	Denmark	1.05	average 1973-1978 (3 pipes)	Olesen
Silt loam	1.4	Neth.	0.23	average 6 sites 1976-1977	Naarding
Silty clay loam	1.3	Neth.	0.19	15 observations 1973-1974	Naarding
Silty clay	1.1	W.Germany	0.39	1 observation 1977-1978	Eggelsman
Clay	1.1	Neth.	0.8	average 1973-1975 (10 pipes)	Naarding

relation between deformation and conductivity can be derived from cores placed in a triaxial apparatus (Boels 3.11). Clay soils with swelling and shrinking properties may recover some of the lost pore space after deformation or compaction (Naarding 3.04; Johansen 3.02).

The hydraulic conductivity in the disturbed zone in the vicinity of the drain pipe can also be derived from measured drain discharge and watertable height. This has been done in fields whose drainage systems were installed either in trenches or with the trenchless method. The ratio between the hydraulic conductivity in the disturbed zone and that in the undisturbed soil are calculated with drainage formulas developed by Ernst (1962). The data presented during the Workshop have been revised and are summarized in Tab. 1.

Table 1 shows that soil disturbance by drain ploughs resulted in a decreased conductivity of the soil. This is true for most types of soil, except for sandy soils. There seems to be a tendency that the deeper the drains are installed, the lower the ratio K_2/K_1 is.

Using the drainage formulas of Ernst, it is possible to compute the additional radial resistance caused by the decrease in hydraulic conductivity of the soil around the pipe. Owing to this extra resistance, the drain spacing of trenchless drainage, L_p, must generally be narrower than the spacing, L, of a trenched system. The ratio between L_p and L can be calculated, assuming the same drainage effectiviness in both cases. From the data in Table 1, the ratios presented in Table 2 are obtained.

TABLE 2. Ratio between the drain spacing for trenchless drainage (L_p) and that for the trenching method (L), assuming equal drainage effectiveness

Soil type	Drain depth (m below surface)	L_p/L
Sand	1.1	1.0
Sandy loam	1.2	0.8
Loam	1.2	1.0
Silt loam	1.4	0.56
Silty clay loam	1.3	0.50
Silty clay	1.1	0.73
Clay	1.1	0.94

This table shows that in most cases the decrease of conductivity in the disturbed zone has only a slight influence on the drain spacing, although under certain conditions it can mean a reduction in spacing of 25 to 50 per cent. The critical depth of the ploughs used in the experiments from which the data in Table 1 stem would be about 1.0-1.1 m or less under the soil conditions met during the drainage operations. The smaller K_2/K_1-ratios and subsequently the small ratios L_p/L could be caused by deformation or compaction in the disturbed zone below the critical depth. If the drain depth was equal to the critical depth, it is most likely that the ratio L_p/L would be one. Trenchless ploughs are not the only cause of low conductivities around the pipe, extremely low conductivities can result with the trenching method when severe soil structure damage occurs or when the trench is backfilled with the soil in a slurry condition.

3.1.4 Depth control and draught requirement

Depth control based on the "floating" plough principle is standard in industry. The term "floating" plough comes from the fact that as the plough is pulled through the soil, the soil-drag forces on the plough blade are in balance with the tractor-draught and plough-gravity forces so that the plough seems to "float" through the soil. Ploughing depth is controlled by changing the attitude or angle of the plough. This can be done by either raising or lowering the real or virtual hitch point or rotating the plough blade.

Because of variations in drag forces on the plough blade, grade control requires more than simply keeping the hitch point height on a line parallel to the desired drain gradient. For precise control, however, information on both the elevation of the plough point and plough attitude is needed. Although this would require two detectors, both factors can be monitored by positioning one detector at 0.833 times the long beam length towards the rear of the plough from the hitch point.

Field-checks of the depth and grade achieved have highlighted a large number of errors. Failures are defined as deviations from the designed depth of more than 0.05 m or when negative slopes occur. Based on a factor analysis, it can be concluded that errors are due less to the type of machine (trenchless or trenching machine) and control system, but more to the soil physical

TABLE 3. Drawbar-pull (kg $\times 10^3$) required as a function of installation depth for various size drain tubes in different kinds of soil

Tube diam.	Clay			Sand gravel			Silt		
	Installation depth (m)			Installation depth (m)			Installation depth (m)		
(m)	1.0	1.5	2.0	1.0	1.5	2.0	1.0	1.5	2.0
0.06	15	22	30	8	14	20	6	9	13
0.11	20	30	40	10	20	30	12	18	25
0.16	25	38	52	15	28	40	15	25	33
0.30	35	52	-	25	40	58	22	36	50

conditions (slope, moisture content, texture, occurrence of stones, topography) and human parameters (particular the machine operator). The best grading results are obtained in uniform soils (loamy soils), followed by clay soils and sandy soils (Cros et al. 3.10). However, it should be noted that little information is available on allowable grading errors in field practice.

Grading accuracy decreases rapidly with increasing forward speed particular at speeds in excess of 0.5-0.75 m/s.

The hitch point, real or imaginary, should be located near the front end of the tractor to bring the resultant force on the prime mover near the centre line of the tracks at the ground surface so as to provide uniform load distribution on the tracks. The track load can thus increase by as much as 30 per cent, thereby increasing the traction-efficiency.

The required drawbar-pull depends upon the width of the plough blade, the working depth, and the kind of soil. Table 3 presents some data.

When the working depth is below the critical depth, the draught increases more rapidly with increasing width of the plough blade than when it is above the critical depth.

The draught increases when the inclination of the tine tip to the horizontal increases. This increase, however, is not large if the angle ranges from 20 to 50°.

3.1.5 Possibilities of deepening the critical depth

The critical depth may be defined as the depth at which the energy required to cause crescent failure equals the energy to cause lateral failure. The energy to cause failure in the soil is determined by the shear strength of the soil. The shear strength is negatively correlated with soil moisture content and positively correlated with the dry bulk density of the soil. The shear strength of a loosened soil is small compared with the initial shear strength.

Loosening the soil surface layer before the deep tining operation results in a deeper critical depth than that obtained in a single-stage operation. If the loosening is done at the same time as the deep tining, the shallow working tines have to be positioned immediately ahead of the deep tine at a distance approximately 1-1.25 times the working depth of the deep tine.

Draught on the deep tine alone is reduced by loosening the top layer. The total draught on a combined shallow/deep tine arrangement is almost the same as that required for a single stage operation with the deep tine alone (Spoor 3.08).

The ways of increasing the critical depth (previously discussed 1.2) are through increasing the plough-width and reducing its inclination to the horizontal at the leading edge.

3.1.6 Considerations concerning the application of the technique

Provided the working depth is less than the critical depth, the trenchless technique can be applied over a wide range of soils without the risk of significant soil damage. Working below the critical depth in coarse-textured soils and perhaps in well-structured peat soils, these machines will not normally impair the effectiveness of drainage. An additional radial flow resistance may occur on fine and medium-textured soils when working below the critical depth, although this resistance may disappear after some years if the soil structure in the deformed or smeared zone can recover.

Compaction or deformation at the side of the drain tube is less important when most of the water enters the pipe from above, e.g. in mole drainage schemes under perched watertables. Working below the critical depth is therefore not so critical in such situations.

The trenchless method requires a much higher draught force than the trenching method. Under conditions with a wet top soil or low bearing capacity, the ploughs can hardly operate without winching spans. Under good topsoil conditions, the potential for high-speed pipe laying is greater with the trenchless method. On stony soils these machines perform more satisfactorily than trenchers.

If the large ploughs are used in drainage schemes larger than about 10 ha, their per metre costs are lower than those of trenchers because of the relatively low wear and tear costs (about 1/3 to 1/5 of those of trenching machines).

Gravel savings are possible where narrower bands of envelopes are acceptable, while in certain cases the reduced milling and pulverisation of the soil may allow tubes to be installed without an envelope, whereas with trenching an envelope would be required.

The trenchless method has some other disadvantages. Existing (old) drains cannot easily be connected to the new-laid system. When composite drainage systems are being installed, a backhoe is always needed to dig holes at the junction between the laterals and the mains.

Severe stretching of plastic pipe can occur at high pipe laying speeds (greater than 0.5-0.75 m per sec.) unless positive pipe feed mechanisms are used.

3.2 Trenching drainage methods

3.2.1 Some experience

The trenching drainage method involves both soil excavation and back-fill operations. Machines can be classified into three main groups: the ladder type (chain-driven buckets), the wheel-digging type, and the digging-chain type. The machines may be equipped with or without a gravel hopper. The digging depth is controlled manually or by some automatic device (e.g. laser equipment).

The machines are suitable for use in most soil types. The chain type machines may work faster than the bucket and wheel types in friable uncemented material, free of rocks and large stones. On cemented soils the bucket type machines and wheel-diggers perform the best.

Under very wet conditions, some chain-type trenchers bring the soil into a state of super saturation, resulting in such a soil consistency that the digging teeth are unable to remove the material from the trench. Filling up the open space between chain and tile box seems to be a solution to this problem.

In saturated unstable soils, the hydrostatic pressure of the soil impairs a proper placing of the gravel envelope. Placing a power auger in the opening around the tube to force the gravel into the open space provided for it seems to overcome the hydrostatic pressure.

3.2.2 Stabilizing unstable trenches

A trench is unstable if yielding in the soil occurs. In principle there are two methods of stabilizing the soil. The first is to reduce the water pressure and the second is to compensate for the overburden pressure.

The first method requires the water table to be drawn down. This can be done by installing first a temporary drain tube of small diameter below the intended depth of the permanent larger drain pipe. Installing an extensive system of well points to drain the immediate vicinity of the drain by pumping will also lower the watertable.

Compensating the overburden pressure can be achieved by over-excavating the bottom and sides of the trench and backfilling it with coarse gravel (Winger 4.04). The trenchless plough technique may also offer advantages in these situations.

3.3 Recommendations

Trenchless drainage installation has some disadvantages. It requires a high draught force and may cause soil deformation or compaction when drains are installed at great depths. Further work is needed on the soil-mechanical aspects of drain ploughs to minimize draught and to determine ways of ensuring that the critical depth is below the required working depth over as wide a range of drainage situations as possible.

Smearing and compaction occur not only with trenchless methods but also with trenching. Further investigations are needed to measure the influence of deformation, compaction, and smearing on the functioning of the drainage system.

As has been reported, some deformed or compacted soils may recover from the ill effects whereas some initially loose soils may consolidate through settling showing a decrease in conductivity. The soil behaviour after drainage has been installed should be monitored to determine the extent of soil recovery or consolidation.

In some papers the need for installing drain tubes at design depth and on design grade has been stressed. However, there is no clear understanding of the influence of installation errors on the functioning of the drainage system.

Investigations are needed to define the importance of these errors and to define working limits.

There is also a need to investigate the factors causing poor grade control with automatic grading systems. These factors include the control system, implement hydraulics and, soil/implement interaction.

Because of the difficulty of checking depth and grade during trenchless installation, a device is required to record these parameters.

The differences in operation costs between trenchless and trenching machines are mainly due to the relatively high wear and tear on the cutting elements of the trenching machines. Improvements should therefore be made to the digging chain and cutters to reduce wear and tear and improve digging efficiency.

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4. DRAINAGE OF IRRIGATED LANDS

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Contents

4.1 Introduction

The total irrigated area in the world has expanded rapidly from 194 million hectares in 1964 to 226 million hectares in 1974. This means an increase of 32 million hectares in 10 years (FAO, 1976). Of that area, 23 million hectares are located in developing countries, especially in China, India, and Pakistan, which have increased their irrigated areas by 15 million hectares.

Irrigation is practised in both dry an humid climates, most of it on alluvial plains and coastal lowlands.

In all climatic regions it is a generally accepted fact that irrigation and drainage are inseparable and that the provision of irrigation should soon be followed by drainage so as to create a suitable root environment for the growth of crops. The reasons for providing drainage are to remove excess soil water, to prevent soil salinization, and to ensure the trafficability and workability of the soil.

4.2 Theory of drainage

Drainage theory is a tool that can be transferred from one place to another. The theory currently available appears to be adequate to tackle most practical design problems. A choice can be made between a number of steady and transient analyses, the choice depending on the type of drainage criterion selected.

Where the drainage criterion aims at a drop in the watertable, which is a common occurrence in humid areas, the transient method is preferred. In arid areas, where salinity control is a common problem and where the drainage criterion will be concerned with the volume of water to be removed, the steady-state method is useful. The most common reason for choosing one method or the other, however, is the availability of input data and the possible definition of criteria.

In practice, both methods are used. The USBR in the USA bases its drainage designs on the transient flow concept. Elsewhere, designs are often based on the steady-state concept, with the results frequently checked later by the transient flow concept. The criteria should eventually be tested in the field and adapted accordingly.

The available drainage formulas for the transient method deal with one-layer soils whereas the formulas for the steady-state method also cover heterogeneous or layered soils. The steady state concept therefore allows more complicated soil hydrological situations to be tackled.

The greatest difficulty in applying any theory is in obtaining adequate input data. The spatial variations in physical parameters are generally far greater than the errors introduced by approximate theories. For example, experience in The Netherlands has shown that the standard deviation of physical parameters is so large that it is difficult to establish representative values. Similar experiences have been reported in the USA, England, and Israel.

Special attention needs to be given to stratified alluvial soils which are heterogeneous both horizontally and vertically. Drainage design formulas are based on one- or two-layer models that assume isotropic soil conditions. They neglect anisotropy resulting from micro-stratification. The introduction of soil anisotropy in drainage design for stratified soils is therefore called for (Boumans, 1.03).

To the extent possible, soil parameters should be measured on a large scale. Practical considerations, however, may force the use of small samples. For example, a test drain may permit the determination of the parameters needed for drainage design on a field scale. Theory should be used not only to design drainage systems but also to derive the appropriate parameters from field tests.

The input parameters for drainage design are difficult to assess. At present only laborious and expensive methods are available to measure them. Improvements are needed in determining large-scale soil parameters in field tests for use in theories, in estimating natural drainage rates, and in the field determination of seepage rates. Soil mapping and drainage design methodology should be better integrated, which could possibly be effectuated by the development of technical land classifications. In this way topographical and geomorphological soil features will receive greater emphasis, thereby allowing the establishment of empirical relations between taxonomical and physical soil characteristics. This could lead to a reduction of actual field determinations by virtue of the greater knowledge of the morphological and physical soil characteristics.

4.3 Drainage criteria

The criterion used for drainage design in irrigated areas defines the position of the watertable as a function of time or the quantity of water to be removed per unit time when the watertable is in a certain position. The criteria imply that the water content and the aeration status of the rootzone are regarded as a direct function of the watertable height. The desired watertable depth differs for the steady and transient approach and further depends on the soil, the crop and the irrigation season. The design recommendations may vary from expert to expert, and appear to be somewhat arbitrary.

The state of knowledge, however, is adequate to establish more reasonable design criteria in most cases. For example, in its drainage designs, the USBR routinely uses the concept of periodic yearly recharge patterns from irrigation, and allows a watertable depth of 0.60 m after pre-irrigation in the peak season and 1.20 m at the end of the irrigation period.

An important consequence of drainage is improved trafficability, which no doubt can be expressed in terms of soil water content. Sufficient information is not yet available to formulate appropriate criteria for trafficability. A combination of surface and subsurface drainage may serve to reduce the trafficability problem on fine-textured soils.

It is mandatory to provide sufficient drainage to prevent, beyond specified limits, the build-up of salts in the soil solution of the rootzone. These limits depend upon the crop, the composition of the irrigation water, and the chemistry of the soil. The amount of drainage required is primarily a function of irrigation management and varies between an upper and a lower limit. The lower limit is determined by the "leaching requirement", which prevents excess build-up of salinity in the rootzone; the upper limit is determined by the ability of the soil to infiltrate water.

Experience has shown that leaching in excess of about 25 per cent is difficult to achieve and may create waterlogging problems. Higher leaching requirements should be avoided by selection of a more salt tolerant cropping pattern. Recent research reveals that the leaching requirements can be reduced considerably below those generally advocated (van Schilfgaarde, 4.03).

Apart from the leaching requirement, a number of other variables should also be considered before a drainage criterion can be established for design purposes. These include the need to distinguish between natural and artificial drainage rates and the effect of a change in cropping pattern. This calls for close interrelation between irrigation management and the drainage need.

To the extent not taken care of by the natural drainage, the required rate of drainage has to be provided by engineering structures. For large projects, say 10 000 ha and more, it may be possible to detect the natural drainage conditions from groundwatertable and groundwater salinity maps, which are often available.

A location with a deep watertable indicates natural drainage and one with a shallow watertable indicates seepage. Likewise a location with low salinity indicates natural drainage and one with high salinity seepage. Both the watertable depth and the degree of salinity can be translated quantitatively into indicators of the natural drainage conditions. This is schematially shown in Table 1, where S_I stands for Seepage Class I, S_{II} for Seepage Class II, D_I for Drainage Class I, D_{II} for Drainage Class II, and 0 for balance.

TABLE 1. Natural drainage conditions related to the depth of the watertable and the groundwater salinity (Boumans, 1976)

Groundwater salinity	Watertable depth		
	deep	medium	shallow
high	0	S_I	S_{II}
average	D_I	0	S_I
low	D_{II}	D_I	0

Under conditions of extensive cropping whereby the land remains fallow for extended periods of time during which there is seepage of groundwater from outside the area, precautions must be taken to avoid the salinization of the rootzone by capillary rise. Where the seepage water is saline, this capillary rise adds greatly to the salt content of the rootzone and particularly of the surface layer.

If seepage flow cannot be eliminated or intercepted, the drain depth should be determined as the sum of the critical watertable depth and the hydraulic head necessary to discharge the seepage flow to the drains. The critical watertable depth is defined as the depth to which the watertable will fall in the absence of seepage and at which capillary rise is reduced almost to zero (van Hoorn, 4.06).

The presence of seepage, however, calls for an identification of its source. As an alternative to providing drainage based on the critical depth criterion, it may be more convenient to reduce, intercept, or eliminate the seepage component.

Whether seepage reduction is a viable alternative to increased drainage intensity is a site-specific question. However, when drainage is seen as an integral part of a total water management system, one arrives at the conclusion that the critical depth should not be used as a fundamental criterion for decision-making.

In areas where drainage is costly, the minimum subsurface drainage should be based on the requirement for salinity control and the evacuation of further excess water via surface drainage. Such a system is applied, for instance, on the vertisols in Morocco.

4.4 Irrigation in relation to drainage

A direct relation exists between irrigation management and the amount of drainage required. Irrigation management has several components, covering on-farm management and the design and management of the distribution system. Increased drainage intensity tends to increase the rate of seepage loss from the distribution system. Poor management causes excessive water losses which add to the amount of water that must be drained.

Even with relatively good management, the amount of deep percolation generally exceeds the leaching requirement. Nevertheless, poor water distribution on the field may well result in inadequate leaching in certain parts of the field, while other parts receive excess water. With the technology at hand at present, substantial improvements in irrigation management are possible.

Field drainage is a curative measure and the high costs of its installation and operation necessitate that the drainage problems be tackled at the points where they are created. This calls for a reduction in the water percolating through the rootzone or leaking from canals and laterals. Any remaining water that adds to the groundwater should be disposed off by an appropriate drainage system.

With the present recorded low irrigation efficiencies, with often only 20 to 40 per cent of the water applied being effectively used for plant growth, wide scope exists for improvements, both in on-farm irrigation and in the distribution system.

Irrigation operations in some countries, particularly in the USA and in Israel, aim at optimum use of energy and water conservation. This can be approached by closed automated irrigation systems, which allow good control over the irrigation water and adequate control of the energy utilization. The management of a closed system on demand is easier than an open irrigation system. The choice between an open and a closed irrigation distribution system is primarily a matter of economics, and farmers technology level, whereby each project has its own approach and its own solution.

The poor quality of operation and maintenance of irrigation systems is often the main cause of low irrigation efficiencies.

Agricultural water management is primarily concerned with on-farm conditions and crop production, but should also be viewed as an important component of natural resource management. Extensive studies in the Colorado River Basin in the USA, for example, have shown that the return flow from irrigated agriculture is a major source of salinity in the Colorado River. This calls for improvements in on-farm water management practices to reduce the saline effluent. This can partly be accomplished by applying the minimum leaching concept as advocated by the US Salinity Laboratory. On the other hand, the productive use of drainage waters of reduced quality could be considered an alternative to disposal. Field studies in the USA have given supporting evidence for the application of both solutions (van Schilfgaarde, 4.03).

4.5 Drainage system and drainage techniques

The drainage system to be applied depends on the nature of the problem and on the environmental conditions. A major distinction can be made between surface drainage systems and subsurface (or groundwater) drainage systems. Two forms of subsurface drainage exist: horizontal and vertical (or tubewell) drainage.

The most common form of drainage for irrigated areas continues to be the parallel horizontal subsurface drainage system. The use of tubewell drainage, however, is spreading in those areas where it is applicable. It has often the double function of draining groundwater and supplying irrigation water.

Drainage costs usually amount to between 30 and 60 per cent of the total costs of an irrigation project, which calls for research and development of more economic systems.

In Iraq a comparison was made of three drainage systems. The first was a singular system of pipe drains, 100 to 250 m long, flowing into an open collector drain; the second was a composite system with a covered pipe as collector; the third was an extended singular system consisting of laterals of greater length than in the other two systems and discharging directly into the main ditch. Of the three, the third was found to offer various advantages for large-scale drainage with wide spacings: it is simple of design and construction, affords an easy check on its proper functioning, and is easy to maintain (Boumans, 4.07).

The selection of the most appropriate system is always a question of economic considerations, although hard economic facts are often lacking. The maintenance costs of an open drainage system, for example, may make it less attractive than a covered system.

Any drainage system layout requires further study in the context of its operation and maintenance. The design should be based not only on theory but also on a basic understanding of the environmental problem. Deserving special attention is the study of surface and subsurface drainage for the improvement of heavy soils.

In the past, research on construction and maintenance techniques has largely been ignored. Wide scope therefore exists for significant improvements.

There has been a tendency towards deeper and more widely spaced farm drains, which calls for large trenching machines. In the USA the machines must meet economic drain depths ranging from 2 to 3 m.

Trenchless drainage with corrugated plastic pipes, 12 cm or less in diameter, is used as an alternative to trenching, down to depths of 1.50 m. Installations to depths of 2.40 m have been reported.

The effect of installation techniques and drainage materials on the radial resistance to flow is the most important single factor in the performance of a drain and should be further studied. This calls for further research on the performance of the various drainage machines (trenching versus trenchless), taking into consideration the required optimum drain depth.

The drainage materials (pipes and envelopes) should keep pace with the rapid technological development in drainage machinery. In the past, clay and concrete pipes have been the most commonly used materials in irrigated areas, but these are now gradually being replaced by corrugated plastic tubing. In many developing countries, however, clay and concrete pipes are still in common use.

Gravel is the main type of drain envelope used in irrigated areas. Research studies indicate that a well-designed, well-graded gravel envelope produces the most water, prevents fines in the base material from moving into the envelope and drain line, and provides the required stability for corrugated plastic drain tubing.

In the USA synthetic envelope materials are only being used where gravel is in short supply and expensive. In permanently irrigated lands, many construction performance requirements are placed on the drainage system. This calls for a clear understanding of the drain construction.

One of the more complex problems is the stabilization of drain trenches when unstable soils are encountered, for instance in soils of silty loam texture with a high groundwatertable. One solution is to use a coarse gravel backfill for stabilization. Another method is to install an extensive system of well points and to evacuate the water surrounding the drain. A third method is to install a plastic drain tube, surrounded by a gravel envelope, below the bottom grade of the permanent drain pipe and to dispose of the effluent into a sump from where it can be pumped into suitable disposal channels.

Proper drainage construction also calls for suitable pipe outlet structures, proper backfill of the drain trenches, and minimum stretching of the corrugated plastic drain tubes.

Proper and timely maintenance of the drainage systems is an absolute must. The main problems with drains in irrigation projects are those associated with unstable soil conditions, with plant roots entering drains and causing plugging, with weed growth in open drain trenches, with improper maintenance of manholes, and with the development of iron and manganese sludges in drains (Johnston, 4.01 and Winger, 4.04).

4.6 Economics of drainage

Large-scale drainage projects must be based on a sound project design and on an evaluation which shows that the benefits will exceed the costs. An economic evaluation of drainage projects, however, is difficult to obtain. Although new development projects are evaluated in a planning stage, the judgements used are only qualitative. A cost calculation is possible, but the benefits of the production enlargement in relation to drainage are more difficult to assess.

The benefits may be divided into direct and indirect benefits and decreased costs. The direct benefits refer to the increases in crop yields, which are not easy to quantify because of their complexity. The effects of lowering the watertable and desalinizing the soil are slow processes and their benefits upon plant growth are not immediately visible.

Because the effect of drainage on crop yield is of such vital importance, an integrated model has been developed in which this effect can be quantitatively computed (Feddes and van Wijk, 1977). In The Netherlands the effects of drainage can be divided into the effects in spring, autumn, and winter, which are mainly those of improved workability of the soil, and the effects in summer, which are determined by the components of the water balance.
The integrated model approach is therefore composed of a workability model describing the conditions during spring, an evapotranspiration model describing the conditions during the summer growing season, and a workability model for the autumn period. The indirect benefits of drainage are agro-economic and socio-economic, and do not enter into an economic evaluation. They may include such benefits as a more efficient employment of labour and a better control of pests and diseases. Indirect benefits often mean that the economic optimum bears little relation to the benefits to society. Drainage projects should therefore be considered in a much wider context. The optimum return from the development and conservation of natural resources should be considered, and the response to drainage worked out on the basis of national welfare.

The data base for economic evaluations is inadequate. As a consequence, economic projections of the benefits of drainage can easily be 20 to 40 per cent out of range. As it is difficult to come up with rational production figures beforehand, the economic analysis in water resources development should not be over-emphasized.

Nevertheless, an economic evaluation of drainage projects should always be made, although the optimization of the ultimate use of natural resources may require decisions that appear to be uneconomic.

4.7 Recommendations

Drainage should not be considered in abstracto but should be treated as an integral part of the total water management system.

The use of drainage theory is too often reduced to the routine application of rules. Each drainage problem requires an individual approach and solution. In training and educational programs, substantial effort should be devoted to developing sufficient understanding of the drainage problems and physical processes involved so that practitioners can use their imagination based on sound principles.

The methodology for field investigations appropriate to local circumstances needs further study. Examples where improvements (or increased uses) are needed include field determination of seepage rates, estimates of natural drainage rates, field tests to determine large-scale soil parameters appropriate for use in theories, and better integration of soil mapping and engineering methodology - possibly by the development of technical land classifications.

To enable irrigation scheduling in accordance with crop demand, serious consideration should be given to designing new irrigation distribution systems or upgrading existing systems, making maximum use of lined channels, reservoirs, closed conduits, and any other changes that enable better system operation.

The concepts that can lead to improved irrigation and drainage practices should be in tune with the attitude and capabilities of the ultimate user. Such concepts will not be adopted unless the knowledge and the limitations of the farmer are considered.

Circumstances need to be identified and technologies refined to make the productive use of drainage waters of reduced quality an effective alternative to disposal.

Maintenance and construction aspects may have far-reaching effects on the drainage design. Further research and discussion on the maintenance of subsurface drainage and its impact on design and construction is needed.

The installation of drainage systems simultaneously with new irrigation systems will generally simplify the installation problems, especially in unstable soils. On the other hand, it complicates the determination of the physical parameters and results in an earlier investment of capital. Careful consideration must be given to the optimization of system installation, taking these conflicting factors into account.

The economic evaluation forms only a part of the assessment of the desirability of a project. Such an evaluation, however, must always be made. Even though the optimization of the ultimate use of natural resources may well require decisions that appear to be uneconomic, such optimization should always be given serious consideration.

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Section II

Contributing papers

Group 100

Design and research

CONTENTS

Paper 1.01

STEADY STATE DRAINAGE IN HETEROGENEOUS AND ANISOTROPIC MEDIA

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Summary

The unconfined aquifer problem is considered in the case of an anisotropic and vertically heterogeneous medium, without making the Dupuit assumptions. Using an auxiliary dependent variable, an equivalent problem is derived, which can be solved exactly for the steady state. Further on, an approximate solution of the initial problem can be deduced.

The method is applied to drainage by open ditches or by tile drains lying upon an impervious layer, eventually with moling superimposed.

An "equivalent horizontal hydraulic conductivity" is derived, which takes the profile heterogeneity into account. This parameter can be estimated "in situ" by pumping tests.

1. General study

1.1 Assumptions and notations

The following assumptions are made:

A Water in the saturated area obeys Darcy's law.

B The soil is anisotropic, the eigen-directions of anisotropy being the natural directions of space (horizontal and vertical). The two horizontal components of hydraulic conductivity are identical.

C The soil is vertically heterogeneous, horizontally homogeneous.

D Soil and water are incompressible.

E The lower boundary of study is an horizontal impervious layer.

F The upper boundary of study is a free surface on which water is at atmospheric pressure.

G The other boundaries of study are vertical.

H The soil is saturated in the studied domain and, immediately above the free surface, water content is equal to saturation, minus a fixed quantity called drainable porosity (or specific yield).

The following notations are used:

D	elevations of moles	(L)
d	downstream hydraulic head	(L)
F	horizontal flow potential	(L^2T^{-1})
h	elevation of the free surface	(L)
i	vertical infiltration rate (positive downwards) crossing the free surface	(LT^{-1})
K_h, K_v	horizontal, vertical hydraulic conductivity	(LT^{-1})
$\bar{K}_h$	equivalent horizontal hydraulic conductivity	(LT^{-1})
L	half-spacing of trenches or drains	(L)
l	half-spacing of moles	(L)
p	effective pressure	$(ML^{-1}T^{-2})$
$\vec{q} = (q_x, q_y, q_z)$	apparent (Darcy's) velocity	(LT^{-1})
r_A	radius of influence (pumping)	(L)
r_p	well radius	(L)
S_y	specific yield or drainable porosity	(1)
$T(z) = \int_o^z K_h dz'$	transmissivity at level z	(L^2T^{-1})
t	time	(T)
$W = \int_o^h S_y dz$	drainable water height	(L)
x, y	horizontal space coordinates	(L)
z	elevation above impervious layer	(L)
ρ	water specific mass	(ML^{-3})
$\phi = z + \frac{p}{\rho g}$	hydraulic head	(L)

1.2 Formulating the general problem

Using the above assumptions, one obtains:

from A and B:

$$q_x = - K_h \frac{\partial \phi}{\partial x} \qquad (1)$$

$$q_y = - K_h \frac{\partial \phi}{\partial y} \qquad \textit{(Darcy's law)}$$

$$q_z = - K_v \frac{\partial \phi}{\partial z}$$

from C:

$$K_h = K_h(z) \quad (2)$$

$$K_v = K_v(z)$$

$$S_y = S_y(z)$$

from D, with no internal sources and sinks:

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial q_z}{\partial z} = 0 \quad \textit{(mass balance)} \quad (3)$$

from E:

$$q_z = 0 \; ; \; z = 0 \quad (4)$$

from F:

$$\phi = z \; ; \; z = h \quad (5)$$

from G: the vertical boundary conditions will be described later

from H:

$$\frac{\partial W}{\partial t} = S_y \frac{\partial h}{\partial t} = - q_x \frac{\partial h}{\partial x} - q_y \frac{\partial h}{\partial y} + q_z + i \; ; \; z = h \quad (6)$$

Formula (6) is a special mass balance equation on the free surface. A proof of it can be found in Wolsack (1976).

1.3 The horizontal flow potential

Charnyi (1951) was the first to introduce a horizontal flow potential for the study of filtration between two ditches. Guyon (1964) has used this concept since 1963 for drainage studies. Here, the potential must be defined as by Zaoui (1964) in order to take vertical heterogeneity into account:

$$F = \int_0^h K_h (\phi - z) \, dz \quad (7)$$

Differentiating (7) with respect to x or y, one obtains:

$$\frac{\partial F}{\partial x} = \int_o^h K_h \frac{\partial \phi}{\partial x} dz + \frac{\partial h}{\partial x} \{K_h (\phi - z)\}_{z=h}$$

$$\frac{\partial F}{\partial y} = \int_o^h K_h \frac{\partial \phi}{\partial y} dz + \frac{\partial h}{\partial y} \{K_h (\phi - z)\}_{z=h}$$

Using (1) and (5):

$$\frac{\partial F}{\partial x} = - \int_o^h q_x dz$$

$$\frac{\partial F}{\partial y} = - \int_o^h q_y dz \qquad (8)$$

Equation (8) shows that the gradient of F equals the negative value of the vertical integral of the apparent velocity. F is thus the horizontal flow potential of the considered problem.

Differentiating again with respect to x or y, one obtains:

$$\frac{\partial^2 F}{\partial x^2} + \frac{\partial^2 F}{\partial y^2} = - \int_o^h \left(\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y}\right) dz$$

$$- \frac{\partial h}{\partial x} \{q_x\}_{z=h} - \frac{\partial h}{\partial y} \{q_y\}_{z=h}$$

Using (3):

$$\frac{\partial^2 F}{\partial x^2} + \frac{\partial^2 F}{\partial y^2} = + \int_o^h \frac{\partial q_z}{\partial z} dz - \frac{\partial h}{\partial x} \{q_x\}_{z=h} - \frac{\partial h}{\partial y} \{q_y\}_{z=h}$$

Evaluating the integral:

$$\frac{\partial^2 F}{\partial x^2} + \frac{\partial^2 F}{\partial y^2} = \{q_z\}_{z=h} - \{q_z\}_{z=0} - \frac{\partial h}{\partial x} \{q_x\}_{z=h} - \frac{\partial h}{\partial y} \{q_y\}_{z=h}$$

Using (4) and (6):

$$\frac{\partial^2 F}{\partial x^2} + \frac{\partial^2 F}{\partial y^2} = \frac{\partial W}{\partial t} - i \qquad (9)$$

1.4 The general method

The general method for obtaining drainage formulas involves the following steps:

(i) Evaluate (exactly or approximately), using (7), the potential F as a function of h on the vertical boundaries and on some remarkable verticals of the considered situation (those on which the free surface is the highest, mainly).

(ii) Solve Dirichlet's problem for F, formulated by (9) and the boundary conditions obtained at step (i).

(iii) Compare expressions of F on the remarkable verticals, as obtained at steps (i) and (ii). One then obtains an approximate formula in which the free surface elevation is connected with the data of the situation.

1.5 The equivalent horizontal hydraulic conductivity

If the pressure distribution on each vertical were hydrostatic (Dupuit's assumption), one would obtain, from (7):

$$F = \int_o^h K_h(z)\,(h - z)\,dz$$

In addition, if the soil were homogeneous, one would have:

$$F = \frac{h^2}{2} K_h$$

From the above considerations, the equivalent horizontal hydraulic conductivity is defined by:

$$\bar{K}_h(z) = \frac{2}{z^2} \int_o^z (z - z')\,K_h(z')\,dz' = \frac{2}{z^2} \int_o^z T(z')\,dz' \qquad (10)$$

It follows that, where the hydrostatic pressure distribution can be assumed as a first approximation of the actual pressure distribution, a first approximation of F will be:

$$F \simeq \frac{h^2}{2} \bar{K}_h(h) \qquad (11)$$

Coming back to a more general situation, a second expression of F can be derived as follows:

integrating (7) by parts, with $\frac{dT}{dz} = K_h$:

$$F = \{T(\phi - z)\}_{z=h} - \{T(\phi - z)\}_{z=h} - \int_o^h T(z)\ (\frac{\partial\phi}{\partial z} - 1)dz$$

using (5), and because T(o) = 0, one obtains:

$$F = \int_o^h T\ (z)(1 - \frac{\partial\phi}{\partial z})dz \qquad (12)$$

combining with (10), (12) can also read:

$$F = \frac{h^2}{2}\ \bar{K}_h(h) - \int_o^h T(z)\ \frac{\partial\phi}{\partial z}\ dz$$

From this, a third expression of F can also be derived:

again integrating by parts, and because (from (10))

$$\frac{d}{dz}\ (\frac{z^2}{2}\ \bar{K}\ (z)) = T(z):$$

$$F + \frac{h^2}{2}\ \bar{K}_h(h) - \{\frac{z^2}{2}\ \bar{K}_h(z)\ \frac{\partial\phi}{\partial z}\}_{z=h} + \{\frac{z^2}{2}\ \bar{K}_h(z)\ \frac{\partial\phi}{\partial z}\}_{z=0} + \int_o^h \frac{z}{2}\ \bar{K}_h(z)\ \frac{\partial^2\phi}{\partial z^2}\ dz$$

hence:

$$F = \frac{h^2}{2}\ \bar{K}_h(h)\ \ (1 - \{\frac{\partial\phi}{\partial z}\}_{z=h}) + \int_o^h \frac{z}{2}\ \bar{K}_h(z)\ \frac{\partial^2\phi}{\partial z^2}\ dz \qquad (13)$$

The (exact) expressions (12) and (13) will be used in the applications to evaluate F approximately on some remarkable verticals.

1.6 Solving Dirichlet's problem

Step (ii) consists in solving the problem formulated by:

$$\begin{aligned} &\frac{\partial^2 F}{\partial x^2} + \frac{\partial^2 F}{\partial y^2} = \frac{\partial W}{\partial t} - i \quad ; \quad (x,y) \in \mathscr{D} \\ &F = F_1\,(x,y) \quad ; \quad (x,y) \in \mathscr{L} \end{aligned} \qquad (14)$$

The classical Green's method is used: introducing a "test-function" G, one obtains:

$$\int_{\mathscr{D}} G\,\left(\frac{\partial^2 F}{\partial x^2} + \frac{\partial^2 F}{\partial y^2}\right) dxdy = \int_{\mathscr{D}} G\,\left(\frac{\partial W}{\partial t} - i\right) dxdy$$

Using Green's identity, one obtains:

$$\int_{\mathscr{D}} F\,\left(\frac{\partial^2 G}{\partial x^2} + \frac{\partial^2 G}{\partial y^2}\right) dxdy = \int_{\mathscr{D}} G\,\left(\frac{\partial W}{\partial t} - i\right) dxdy - \int_{\mathscr{L}} \left(G\,\frac{\partial F}{\partial n} - F\,\frac{\partial G}{\partial n}\right) d\sigma(x,y) \qquad (15)$$

Taking G equal to the Green's kernel of point (x',y'), i.e. the solution of:

$$\begin{aligned} &-\left(\frac{\partial^2 G}{\partial x^2} + \frac{\partial^2 G}{\partial y^2}\right) = \delta(x - x')\,\delta\,(y - y') \quad ; \quad (x,y) \in \mathscr{D} \\ &G = 0 \quad ; \quad (x,y) \in \mathscr{L} \end{aligned} \qquad (16)$$

One obtains, combining (14), (15) and (16):

$$F(x',y') = \int_{\mathscr{D}} G\,\left(i - \frac{\partial W}{\partial t}\right) dxdy - \int_{\mathscr{L}} F_1\,\frac{\partial G}{\partial n}\,d\sigma(x,y) \quad ; \quad (x',y') \in \mathscr{D} \qquad (17)$$

For the applications (except for the pumping test), one will use (17) with an appropriate choice of (x',y'). For the pumping test, one will use (15).

2. Applications (steady state)

2.1 Trenches attaining the impervious layer

The flow is invariant with any translation parallel to $\vec{Oy}$. Space coordinates x and z only are to be used.

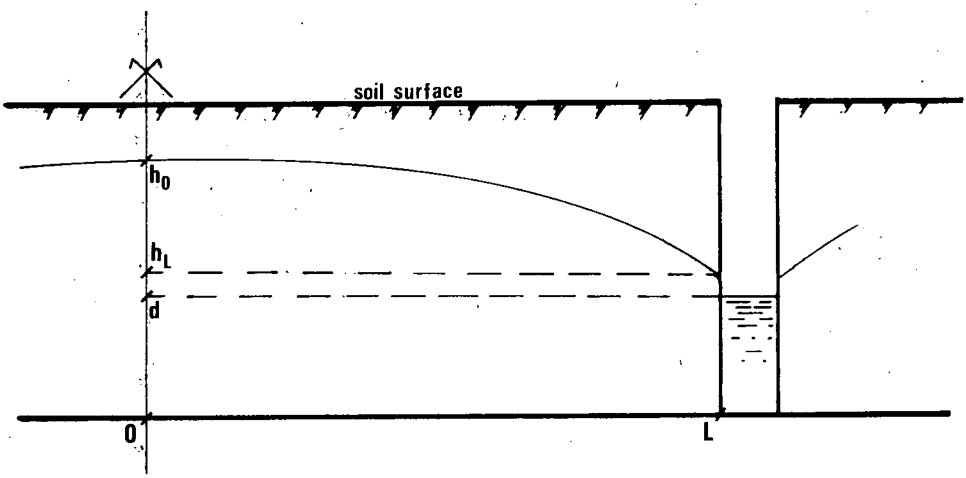


Fig. 1. Trenches reaching the impervious layer.

One denotes (Fig. 1) L the half-spacing of the trenches and d the water level in the trenches. The free surface is the highest at x = 0 (elevation h_o). The boundary conditions are:

$$\phi = d \quad ; \quad 0 < z < d \quad ; \quad x = \pm L \tag{18}$$

(i) One has, from (7), (10) and (18):

$$F(-L) = F(L) = \frac{d^2}{2}\,\overline{K_h}(d) \tag{19}$$

At x = 0, one has, from (12):

$$F(0) = \int_0^{h_o} T(z)\,\left(1 - \frac{\partial\phi}{\partial z}\right) dz \tag{20}$$

q_z is necessarily negative at x = 0. Then, from (1), $\frac{\partial\phi}{\partial z}$ is positive. In addition, $\frac{\partial\phi}{\partial z}$ is in general maximum on the free surface (the contrary implies existence of a very poorly permeable layer, which would have been considered as impervious). It follows from (6) that, in general:

$$0 < \frac{\partial\phi}{\partial z} < \frac{i}{K_v(h_o)} \qquad ; \quad x = 0$$

Substituting into (20) and combining with (10):

$$\frac{h_o^2}{2} \overline{K_h} (h_o)(1 - \frac{i}{K_v(h_o)}) < F (o) < \frac{h_o^2}{2} \overline{K_h}(h_o) \tag{21}$$

One can then write, by analogy with Guyon-Thirriot (1967):

$$F (0) = \frac{h_o^2}{2} \overline{K_h}(h_o) (1 - 2 \frac{i\mathcal{R}_o}{K_v(h_o)}) \tag{22}$$

with $0 < \mathcal{R}_o < \frac{1}{2}$ (last inequality only probable)

(ii) Taking $G = \frac{1}{2} |L - x|$, it comes from (17):

$$F (0) = i \frac{L^2}{2} + \frac{1}{2} \{F (L) + F (- L)\} \tag{23}$$

(iii) Combining (19), (22) and (23):

$$i L^2 = h_o^2 \overline{K_h} (h_o) (1 - 2 \frac{i\mathcal{R}_o}{K_v(h_o)}) + d^2\overline{K_h} (d) \tag{24}$$

The corresponding formula for the homogeneous and anisotropic case is (Guyon & Thirriot, 1967):

$$iL^2 = h_o^2 K \left(1 - 2 \frac{i \mathcal{R}_o}{K}\right) + d^2 K$$

2.2 Drains laying on the impervious layer

Assuming the trench effect (Trench backfill highly permeable, when compared to natural soil), the situation is the same as in Section 2.1, with d = 0 (Fig.2).

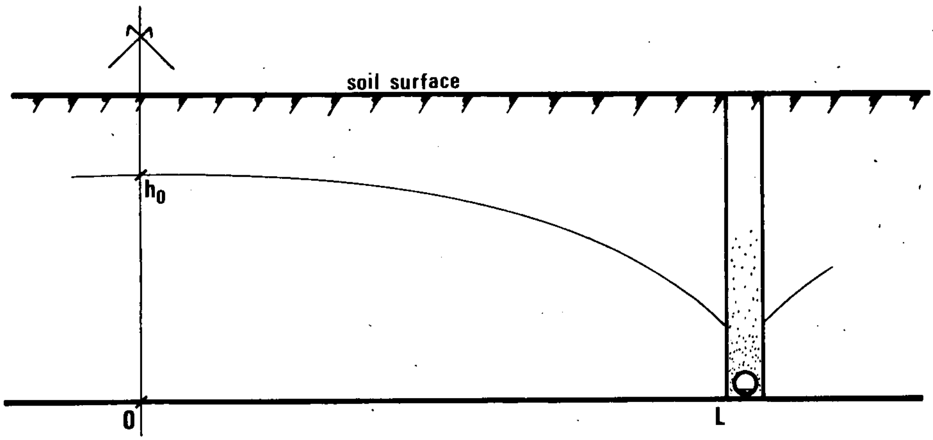


Fig.2. Drains laying on the impervious layer.

One directly obtains from (24):

$$i\,L^2 = h_o^2 \; \overline{K_h}\,(h_o)\; \left(1 - 2\,\frac{i\,\mathcal{R}_o}{K_v(h_o)}\right) \qquad (25)$$

2.3 Drains laying on the impervious layer, with moles superimposed

The $\vec{Ox}$ axis is chosen perpendicular to drains, the half-spacing of which is denoted by L (Fig.3).

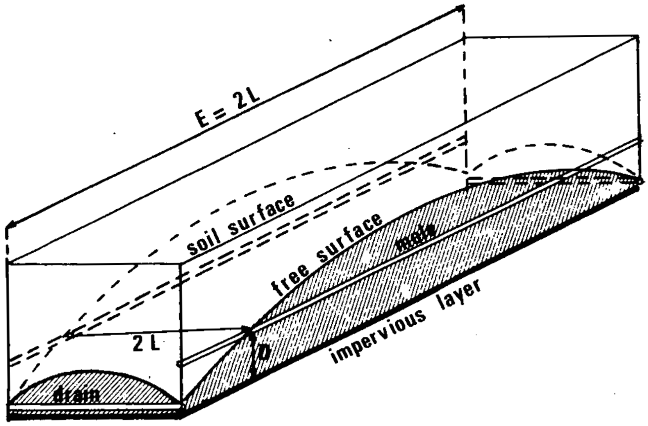


Fig.3. Drains laying on the impervious layer, with moles superimposed (perspective view).

The $\vec{Oy}$ axis is chosen perpendicular to moles (supposed perpendicular to drains). One denotes by l their half-spacing and by D their elevation. The free surface is the highest at point (0,0 - elevation h_c).

(i) The boundary condition on the drain verticals is:

$$\phi = 0 \; ; \; x = \pm L \; ; \; 0 < z \qquad \textit{(trench effect)} \tag{26}$$

Thus:

$$F(\pm L, y) = 0 \tag{27}$$

On the mole verticals the situation is more complex. In the vicinity of drains, the elevation h (x, ±1) of the free surface is smaller than D. Thus:

$$\frac{\partial \phi}{\partial y} = 0 \; ; \; y = \pm 1 \; ; \; 0 < z < h\ (x, \pm 1) \qquad (28)$$

Far from the drains, h (x, ± 1) is greater than D (at least when the water table is high) and the condition is:

$$\frac{\partial \phi}{\partial y} = 0 \; ; \; y = \pm 1 \; ; \; 0 < z < D \qquad (29)$$

$$\phi = z \; ; \; y = \pm 1 \; ; \; D < z < h\ (x\ , \pm L) \quad \textit{(trench effect above mole)}$$

Midway from the drains (at x = 0), water is flowing upwards (at least partially), but this flow is certainly sufficiently weak so that ϕ does not exceed much D. Thus, one has, from (7) and (10):

$$F\ (0, \pm 1) \simeq \frac{D^2}{2} K_h\ (D) \qquad (30)$$

It follows that, on the verticals of moles, F is varying from $\frac{D^2}{2} K_h$ (D) at x = 0 to 0 at x = ± L. In order to get simple calculations, one assumes that this variation is sinusoïdal:

$$F\ (x, \pm 1) \simeq \frac{D^2}{2} \overline{K_h}\ (D) \cos\ (\frac{\Pi}{2} \frac{x}{L}) \; ; \; - L < x < + L \qquad (31)$$

Finally, at (0,0), one has, as at x = 0 in situation 2.1:

$$F\ (0,0) = \frac{h_c^2}{2} \overline{K_h}\ (h_c)\ (1 - 2 \frac{i \mathcal{R}_c}{K_v (h_c)}) \qquad (32)$$

$$\text{with } 0 < \mathcal{R}_c < \frac{1}{2}$$

(ii) Taking G as Green's kernel of point (0,0), say:

$$G = \frac{8}{\Pi L l} \sum_{pq=o,\infty} \frac{1}{k_{pq}} \cos\ \{(2p + 1) \frac{\Pi x}{2L}\} \cos\ \{(2q + 1) \frac{\Pi y}{2l}\}$$

$$k_{pq} = \frac{(2p + 1)^2}{L^2} + \frac{(2q + 1)}{l^2} \qquad (33)$$

(iii) *Combining (27), (31) and (33), one obtains, after some manipulations:*

$$2\ \mathrm{ILl}a\left(\frac{l}{L}\right) = h_c^2\ \overline{K_h}\,(h_c)\ \left(1 - 2\,\frac{i\,\mathcal{R}_c}{K_v(h_c)}\right) - D^2\ \overline{K_h}\,(D)\ b\left(\frac{l}{L}\right) \tag{34}$$

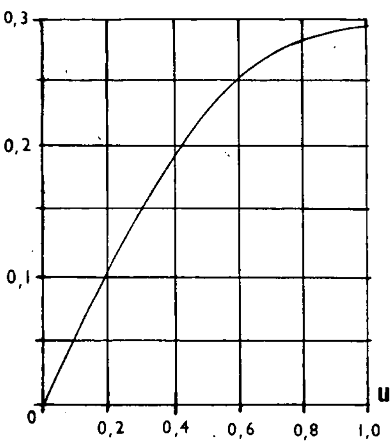
with

$$a\ (u) = \frac{64}{\Pi^4} \sum_{pq=o,\infty} \frac{(-1)^{p+q}}{(2p+1)(2q+1)}\ \frac{1}{((2p+1)^2 u + (2q+1)^2/u)} \tag{35}$$

(see Fig.4)

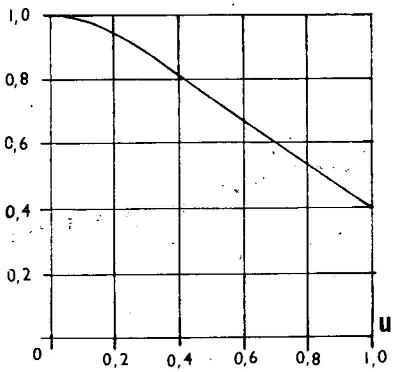
and

$$b\ (u) = \frac{4}{\Pi} \sum_{q=o,\infty} \frac{(-1)^q (2q+1)}{u^2 + (2q+1)^2} \qquad \textit{(see Fig.5)}$$



u	b	u	b
0	0	0.16	0.080
0.01	0.005	0.17	0.085
0.02	0.010	0.18	0.090
0.03	0.015	0.19	0.095
0.04	0.020	0.20	0.100
0.05	0.025	0.30	0.148
0.06	0.030	0.40	0.192
0.07	0.035	0.50	0.228
0.08	0.040	0.60	0.255
0.09	0.045	0.70	0.274
0.10	0.050	0.80	0.286
0.11	0.055	0.90	0.293
0.12	0.060	1.00	0.295
0.13	0.065		
0.14	0.070		
0.15	0.075		

Fig.4. Coefficient a as a function of $u = \frac{l}{L}$



u	b	u	b
0	1	0.16	0.969
0.01	0.9998	0.17	0.965
0.02	0.9994	0.18	0.961
0.03	0.9988	0.19	0.957
0.04	0.0079	0.20	0.953
0.05	0.9968	0.30	0.898
0.06	0.9954	0.40	0.830
0.07	0.994	0.50	0.755
0.08	0.992	0.60	0.676
0.09	0.990	0.70	0.599
0.10	0.988	0.80	0.526
0.11	0.985	0.90	0.459
0.12	0.982	1.00	0.398
0.13	0.979		
0.14	0.976		
0.15	0.973		

Fig.5. Coefficient b as a function of $u = \frac{l}{L}$

2.4 Well reaching the impervious layer

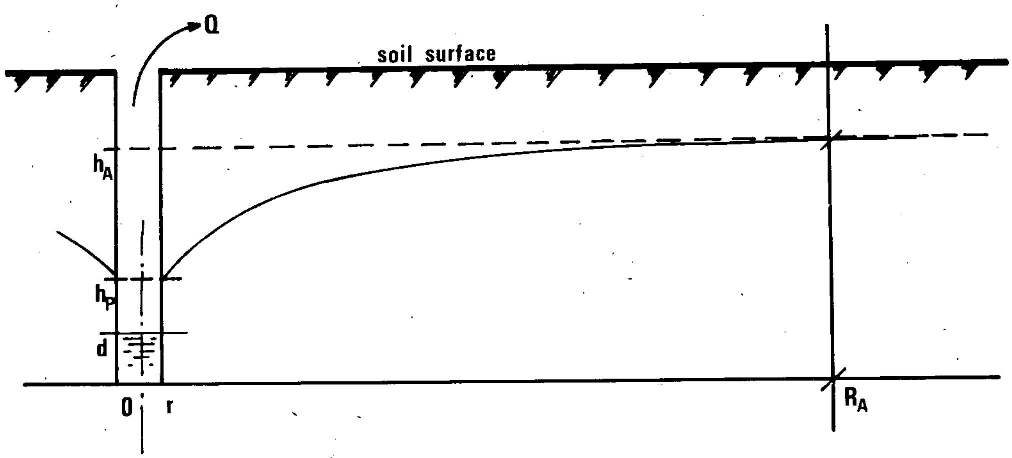


Fig.6. Well reaching the impervious layer.

The flow is invariant by any rotation around the well axis. The horizontal coordinate $r = (x^2 + y^2)^{\frac{1}{2}}$ (distance to well-axis) is to be used. One denotes r_p the well-radius, d the water level in the well and h_p the free surface elevation at $r = r_p$. One denotes r_A a distance to the axis much greater than r_p (of the order of magnitude of the pumping radius of influence) and h_A the free surface elevation in $r = r_A$. There is no infiltration.

(i) The boundary condition at $r = r_p$ is:

$$\phi = d \; ; \quad 0 < z < d \; ; \quad r = r_p \tag{36}$$

$$\phi = z \; ; \quad d < z < h_p \; ; \quad r = r_p$$

Then, from (7) and (10):

$$F\,(r_p) = \frac{d^2}{2}\; \overline{K_h}\,(d) \tag{37}$$

At $r = r_A$, one has, from (12):

$$F\,(r_A) = \int_0^{h_A} T\,(z)\;(1 - \frac{\partial \phi}{\partial z})\; dz$$

The greater r_A , the more the pressure distribution approaches the hydrostatic distribution. One writes, by analogy with (22):

$$F\,(r_A) = \frac{h_A^2}{2}\; \overline{K_h}\,(h_A)\;(1 + 2\,\frac{q_z(r_A,h_A)}{K_v(h_A)}\,\mathcal{R}_A)$$

The value of $\mathcal{R}_A$ cannot be assessed because the pressure distribution is very variable due to the profile heterogeneity. In addition, $q_z\,(r_A,\,h_A)$ cannot be simply estimated.

One will neglect here the corrective term, and obtain then:

$$F\ (r_A) \simeq \frac{h_A^2}{2}\ \overline{K_h}\ (h_A) \tag{38}$$

(ii) Taking $G = \ln\ (\frac{r}{r_A})$, *(15) yields:*

$$2\Pi r_a \frac{dF}{dr}\Big|_{r\ =\ r_A} \ln\ (\frac{r_A}{r_p}) = 2\Pi\ \ (F\ (r_A) - F\ (r_p)) \tag{39}$$

Denoting Q the pumping discharge, one has, from (8):

$$Q = 2\Pi r_A \frac{dF}{dr}\Big|_{r=\ r_a} \tag{40}$$

(iii) Combining (37), (38), (39) and (40), one finally obtains:

$$\frac{Q}{\Pi} \ln\ (\frac{r_A}{r_p}) \simeq h_A^2\ \ \overline{K_h}\ (h_A) - d^2\ \overline{k_h}\ (d) \tag{41}$$

The corresponding formula for the homogeneous and anisotropic case is (Guyon, 1971):

$$\frac{Q}{\Pi} \ln\ (\frac{r_A}{r_p}) \simeq (h_A^2\ - d^2)K$$

3. Conclusions

Taking the vertical heterogeneity in account does not present major difficulties in the applications considered here, i.e. in steady state and with trenches, drain pipes, or well reaching the impervious layer.

In these situations, the profile heterogeneity is completely described at first approximation with the concept of "equivalent horizontal hydraulic conductivity". The major advantage of this concept is that the classical formulas for homogeneous cases can still be used, with only a slight modification: one has just to replace the homogeneous hydraulic conductivity K by the equivalent horizontal hydraulic conductivity $\overline{K_h}$ in these formulas. However, the corrective terms of the formulas, which are generally neglected in practice, would be more difficult to evaluate in the heterogeneous case.

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Paper 1.02

SECOND AND THIRD DEGREE EQUATIONS FOR THE DETERMINATION OF THE SPACINGS BETWEEN PARALLEL DRAINAGE CHANNELS

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Summary

The drainage formula proposed by Hooghoudt can be combined with one proposed by the author. This results in a third degree polynomial equation for the drain spacing. The resulting formula can be applied to the steady state groundwater flow to parallel drains in homogeneous aquifers and in two-layer aquifers with the interface at the same level as the open water surface in the drainage channels. For some three-layer aquifers the same equation can be used in combination with a nomograph for the radial flow resistance in a two-layer aquifer. An attempt has been made to obtain a simpler formula by adding an empirical coefficient to the radial flow resistance. The result is a slightly less accurate second degree equation. A comparison of these formulae with the results of Kirkham and Toksöz did show only small differences. The formulae presented in this paper can therefore safely be used for practical applications, however with the condition that the drain spacing must be at least equal to four times the total thickness of the aquifer.

1. Drainage of a homogeneous aquifer

In The Netherlands two formulae are rather widely used for the computation of discharge, drain spacing or phreatic level. Both formulae take into account the radial flow to the parallel, horizontal drains.

The older formula of these two is obtained by assuming a completely horizontal flow, but instead of the thickness H_o of the aquifer between drain level and impermeable base (Fig.1a), a reduced thickness d (Fig.1b) has to be introduced to take into account the influence of the radial flow (Hooghoudt, 1940).

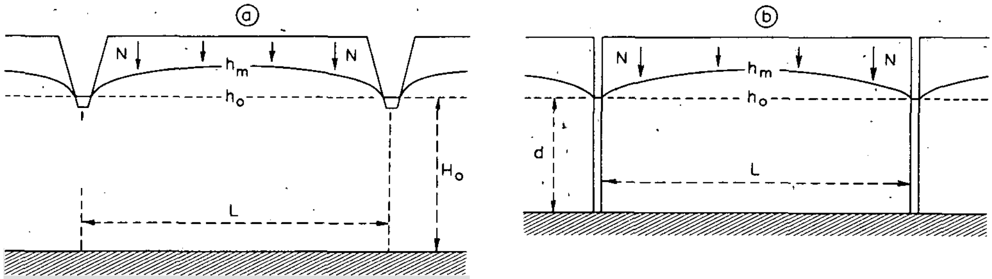

Fig.1. Steady state groundwater flow in a homogeneous aquifer. (a) Real situation with partially penetrating parallel drains. (b) Imaginary situation with fully penetrating drains. Inflow, outflow, drain spacing and potential difference are supposed equal in both cases. The equivalent layer must have a thickness $d < H_o$.

$$N = U = \frac{4k(h_m-h_o)^2 + 8kd(h_m-h_o)}{L^2} \qquad (1)$$

where

- N = precipitation surplus P-E (in steady state equal to downward flux through phreatic surface)
- U = discharge by drains per unit of horizontal area
- k = hydraulic conductivity
- L = spacing between parallel drains
- h_m = hydraulic head of the groundwater midway between the drains
- h_o = hydraulic head of the open water in the drains
- d = thickness of equivalent layer

The parameter d depends solely on the thickness H_o of the aquifer below drain level, the drain spacing L and the wet perimeter B_{wp}. Hooghoudt used infinite series to compute the parameter d (see tables in the original paper; Hooghoudt, 1940). These infinite series can be replaced by a closed expression containing hyperbolic functions (Labye, 1960). In spite of this simplification d is a rather complicated function of L, so that an explicit solution of L cannot be obtained in this way. However, equation (1) has the advantage, that it shows immediately that there is a second degree relation between h_m-h_o and N or U.

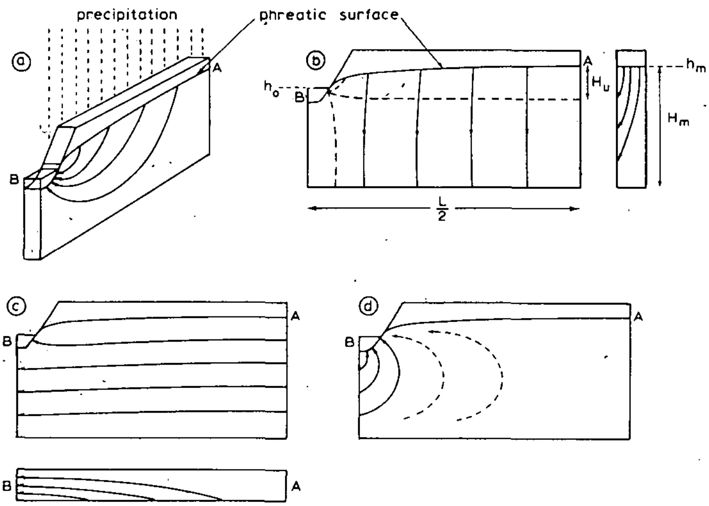

Fig.2. Separation of the groundwater flow into three components for vertical, horizontal and radial flow: $a = b + c + d$

Another formula valid for the same situation has been obtained by distinguishing a vertical, a horizontal and a radial component in the flow from land surface to drainage channel (see b, c and d in Fig.2 and Ernst, 1956, 1962, 1963). The potential difference between A and B has to be determined for each of the Figures 2b, c and d. Addition will give the potential difference valid for Fig.2a:

$$h_m - h_o = \Delta h_{vert} + \Delta h_{hor} + \Delta h_{rad} \tag{2}$$

with

$$\Delta h_{vert} = \frac{N}{k}\left(H_u - \frac{H_u^2}{2H_m}\right) \simeq \frac{N}{k}(h_m - h_o) \tag{3}$$

$$\Delta h_{hor} = \frac{NL^2}{4k(H_o + H_m)} = \frac{NL^2}{8kH_{av}} \tag{4}$$

$$\Delta h_{rad} = NL\Omega \tag{5}$$

where

H_o = thickness of the aquifer between drain level and impermeable base

H_{av} = average thickness of aquifer

Ω = radial flow resistance

By substitution of Eqs. 3, 4 and 5 into Eq. 2:

$$h_m - h_o = \frac{N}{k}(h_m - h_o) + \frac{NL^2}{8kH_{av}} + NL\Omega$$

or

$$\frac{k-N}{k}(h_m - h_o) = \frac{NL^2}{8kH_{av}} + NL\Omega \tag{6}$$

Eq. 6 seems to be quite close to a linear relation between $h_m - h_o$ and N, because in nearly all practical cases the coefficient $(k-N)/k$ will lie between 0.9 and 1. However, this is not a reason to consider Eq.6 very different from the second degree Eq.1. It should be born in mind, that not only H_{av} is depending on $h_m - h_o$, i.e. $H_{av} = H_o + \frac{1}{2}(h_m - h_o)$, but that there is also a slight decrease in the radial flow resistance Ω for increasing discharge.

For the moment all non-linear effects will be discarded by assuming that only situations with small N/k and consequently small $h_m - h_o$ have to be dealt with. This implies that Eq.6 can be replaced by:

$$h_m - h_o = \frac{NL^2}{8kH_o} + NL\Omega_o \tag{7}$$

with

Ω_o = radial flow resistance for a nearly horizontal phreatic surface

For a drainage channel with a half circular wet perimeter or with a width about equal to twice its depth (Fig. 3a), the following expression can be used (Ernst, 1962):

$$k\Omega_o = \frac{1}{\pi} \ln \frac{H_o}{B_{wp}} \tag{8}$$

where

B_{wp} = wet perimeter of the drainage channels

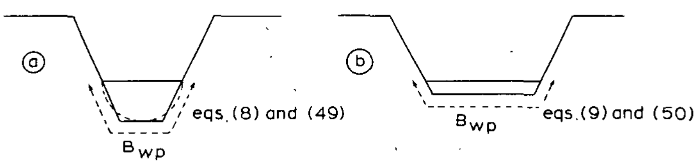

Fig.3. Drainage channels with trapezoidal wet cross-sectional area.

(a) Width about equal to twice the depth, which case is fairly equivalent to a half circular shape. In the homogeneous aquifer Eq.8 can be used. In case of a two layer aquifer see Fig.9 and Eq.50.

(b) Width much larger than depth, which case is about equal to a zero depth. For a homogeneous aquifer and a two layer aquifer the Eqs.9 and 51 can be used respectively.

When the depth of the channel is small compared with the width (Fig.3b), the radial flow resistance Ω_o can be determined by means of (Ernst, 1962):

$$k\Omega_o = \frac{1}{\pi} \ln \frac{4H_o}{\pi B_{wp}} \tag{9}$$

The decrease of Ω, with an increasing discharge q_o = NL through each of the drainage channels, is a rather complicated problem, which has not been investigated thoroughly up to now. For a homogeneous aquifer both the depth H_o of the impermeable base, the shape of the drainage channel (e.g.: slope α, see Fig.4) and the discharge intensity q_o should be taken into account.

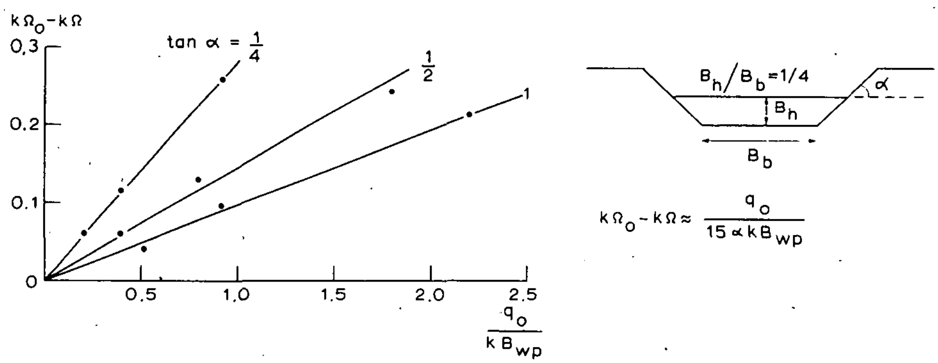

Fig.4. Nomograph for the decrease in radial flow resistance Ω with increasing discharge q_o for a wet cross-sectional area as shown in the right hand side of this figure (Ernst, 1962, Fig.28c). The water depth is assumed to remain constant.

The magnitude of the decrease of Ω can be read from Fig.4. Because in most practical cases $q_o/kB_{wp} < 1$, it can be assumed that this increase is not very important except for flat slopes and very large discharge intensities.

Another question which has still to be discussed is the applicability of the preceding formulae on very thick aquifers. Both expressions 8 and 9 are not to be used in case of very large H_o/L-values. It can be seen immediately that use of these equations for $H_o = \infty$ would lead to $h_m - h_o = \infty$. It is obvious, that for increasing H_o there must be a gradually decreasing hydraulic head difference $h_m - h_o$, with the following minimum values for the cases corresponding to Eqs.8 and 9 (Ernst, 1956, 1962):

$$H_o \to \infty \nearrow \quad h_m - h_o \to \frac{NL}{\pi k} \ln \frac{L}{B_{wp}} \tag{10}$$

$$H_o \to \infty \searrow \quad h_m - h_o \to \frac{NL}{\pi k} \ln \frac{4\ L}{B_{wp}} \tag{11}$$

Substitution of Eqs.8 or 9 in 7 and comparison with Eq.10 or 11 shows that the accuracy of these equations is satisfactory if $H_o < L/4$. Formulae containing radial flow resistances can therefore be accepted especially for those practical problems in which H_o/L and $(h_o - h_m)/L$ have no excessive values.

2. A third degree equation for the drain spacing

Still using the assumption that N/k and $h_m - h_o$ are relatively small, Eq. 1 can be simplified to:

$$h_m - h_o = \frac{NL^2}{8kd} \tag{12}$$

From 7 and 12 it follows immediately that

$$\frac{d}{H_o} = \frac{L}{L + 8kH_o\Omega_o} \tag{13}$$

This expression for d can also be used in Eq.1 without neglecting the second degree term. Then only one condition has to be obeyed: $H_o < L/4$. For larger values of H_o Eq. 13 can still be used by introducing a fictitious value H_* being about one fourth of the presumable value of L.

Substitution of Eq. 13 into 1 gives:

$$NL^2 = 8kH_o \frac{L}{L + 8kH_o\Omega_o} (h_m - h_o) + 4k(h_m - h_o)^2 \tag{14}$$

or:

$$\frac{NL^2}{16kH_o^2} = \frac{L(h_m-h_o)}{(L + 8kH_o\Omega_o)2H_o} + \left(\frac{h_m-h_o}{2H_o}\right)^2 \tag{15}$$

Eq.14 contains 7 variables: N, L, k, H_o, Ω_o, h_m and h_o. By re-arranging this formula as in Eq.15 it can be seen immediately that the present relation can be considered as containing only 4 dimensionless parameter combinations:

$$\frac{N}{k}, \frac{L}{H_o}, \frac{h_m-h_o}{H_o} \text{ and } k\Omega_o$$

This number can be reduced to 3, by substituting of the following parameters λ, β_1 and γ. Each of these parameter combinations contains only one of the most important properties of the system: drain spacing $L \rightarrow \lambda$, hydraulic head difference $h_m-h_o \rightarrow \beta_1$, precipitation rate $N \rightarrow \gamma$.

$$\frac{L}{8kH_o\Omega_o} = \lambda \tag{16}$$

$$\frac{h_m-h_o}{2H_o} = \beta_1 \tag{17}$$

$$2k\Omega_o\sqrt{\frac{N}{k}} = \gamma \tag{18}$$

Substitution of Eqs. 16, 17 and 18 in 15 results in:

$$\gamma^2\lambda^2 = \frac{\lambda}{\lambda+1}\beta_1 + \beta_1^2 \tag{19}$$

Eq.19 is a third degree equation in λ, like Eqs.14 and 15 are third degree equations in L. A rather convenient method for a solution of λ is making use of the nomograph shown in Fig.5.

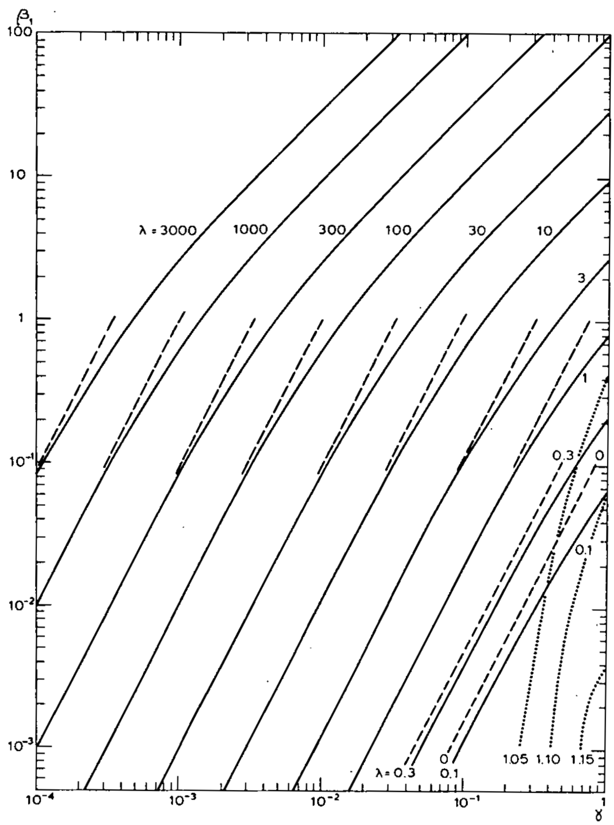

Fig.5. Nomograph for the determination of the drain spacing.

——— Eq.(19) -------- Eq.(20) 1.05;

$$1.10 \text{ and } 1.15 = \frac{\lambda \text{ acc. to Eq.18}}{\lambda \text{ acc. to Eq.41}}$$

For any arbitrary combination of β_1 and γ the corresponding λ-value can be read directly from this nomograph.

Finally Eq.6 has to be used to obtain the value of L satisfying the original Eq.14.

By the straight, broken lines in Fig.5, it can be seen that the following approximation proposed by Van Beers (1978) is a very good one, provided

$\beta_1 < 0.1$ and $\lambda > 1$.

$$(\lambda + \frac{\pi}{8})\gamma = \sqrt{\beta_1} \tag{20}$$

or

$$\frac{L}{H_o} + \pi k\Omega_o = \sqrt{\frac{8k(h_m-h_o)}{NH_o}} \tag{21}$$

In view of the fact that there is a large range of conditions for which the accuracy of Eqs.20 and 21 is not satisfactory, these equations will not be considered furthermore in this paper.

The increasing elevation of the phreatic surface with increasing precipitation must result in a non-linear relation between N or U and h_m-h_o. This has been taken into account by Hooghoudt (Eq.1) by assuming that the flow in the ground above drain level is the main reason for the non-linear behaviour. Neglect of the vertical component of the flow above drain level is not always allowed. The vertical flow is of importance if $0.2 < N/k < 1$ and also in two-layer aquifers with a rather small hydraulic conductivity in the upper region. Whatever the importance may be, the vertical flow component can be taken properly into account by addition of a coefficient $k/(k-N)$, as has been shown by Eqs. 3 and 6 (Ernst, 1956; Kirkham, 1961).

It is obvious that this coefficient should also be added to Eqs. 1 and 14 respectively resulting in:

$$N = \frac{4(k-N)(h_m-h_o)^2 + 8d(k-N)(h_m-h_o)}{L^2} \tag{22}$$

and

$$\frac{N}{k-N} L^2 = \frac{L}{L + 8kH_o\Omega_o} \cdot 8H_o(h_m-h_o) + 4(h_m-h_o)^2 \tag{23}$$

Eq.19 remains valid. Only the expression for the parameter γ has to be changed a little:

$$\gamma = 2k\Omega_o \sqrt{\frac{N}{k-N}} \tag{24}$$

In the analysis achieved by Kirkham and Toksöz (Kirkham, 1958; Toksöz and Kirkham, 1971a and 1971b) the horizontal flow above drain level has been neglected, which implies that $d^2h/dN^2 > 0$. This involves that a comparison of these results with Eq. 23 should be done in the first place for $N << K$. Under this condition Kirkham's formula (Kirkham, 1958) and Eq.23 will give practically equal results except for $H_o > L/4$, where the combination of Eq.7 with 8 or 9 is failing.

3. The inflection point in the $h_m(N)$-relation

In many practical cases a linear relation between N=U and $h_m - h_o$ can be assumed without the implication of large errors. Introduction of special assumptions for the flow direction above drain level - horizontal flow or vertical flow - will result in non-linear relations with exclusively negative or exclusively positive values for the second derivative d^2h/dN^2.

In the preceding chapter it has been shown, that from a fundamental point of view a more satisfactory relation can be obtained by means of the third degree Eq.23, with both positive and negative values of the second derivative d^2h/dN^2. The conditions under which for practical application a linear or non-linear relation might be assumed, can be most easily discussed by writing equation 22 in a slightly different way:

$$4\left[\left(\frac{h_m - h_o + d}{L}\right)^2 - \left(\frac{d}{L}\right)^2\right] = \frac{N}{k-N} \tag{25}$$

Equation 25 can also be written as:

$$4(y^2-a^2) = \frac{x}{1-x} \tag{26}$$

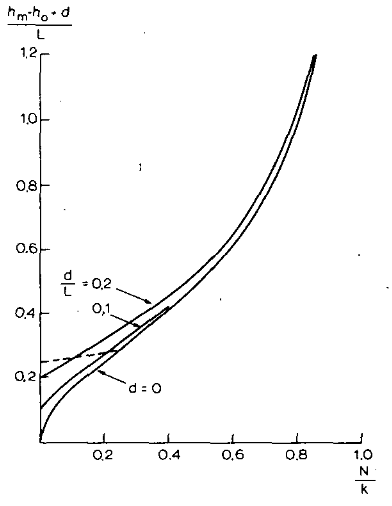

Fig. 6. Relation between hydraulic head h_m-h_o and precipitation surplus N dependent on d, L and k according to Eq.25.
- - - - - - locus of inflection points.

In each inflection point the second derivative must be equal to zero:

$$\frac{d^2y}{dx^2} = 0 \tag{27}$$

By elimination of a, from Eqs. 26 and 27, the locus of the inflection points is found to be:

$$\frac{h_m-h_o + d}{L} = \frac{1}{4}\sqrt{\frac{k}{k-N}} \tag{28}$$

Eq.25 and the locus of inflection points according to Eq.28 are shown in Fig.6. It can be seen that a marked curvature is only possible for relatively large values of $(h_m-h_o)/L$ or (and) d/L. According to Hooghoudt's tables the maximum value of $d/L = 0.34$. Values of $(h_m-h_o)/L$ about equal to 0.1 and larger do not exist under practical conditions. This implies that the influence of $k/(k-N)$ will seldom be so large that the concave curvature will be predominating. The convex curvature, which follows from the Hooghoudt equation 1, is only of importance for relatively small values of d/L. This cannot always be considered to be neglectable, especially for two-layer aquifers with $k_1 << k_2$, which will be considered in the next chapter.

4. Two-layer aquifers

The heterogeneous aquifer, to which Hooghoudt's formula can be applied equally well as to the homogeneous aquifer, is made up of two layers with permeabilities k_1 and k_2 **and divided by a horizontal boundary running** through the level of the open water in the drainage channels (Fig.7). For this case Eq.1 can be changed into:

$$N = U = \frac{8k_2 d(h_m-h_o) + 4k_1(h_m-h_o)^2}{L^2} \qquad (29)$$

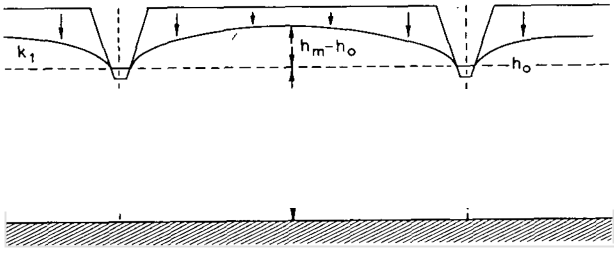

Fig.7. Groundwater flow to partially penetrating drains. Situation comparable with Fig.1a, but in this case a two-layer aquifer with hydraulic conductivities k_1 and k_2, respectively above and below the level of the open water.

In Chapter 1 the vertical component of the flow above the drain level has been taken into account by Eq. 3. The potential difference for the vertical flow in the upper layer can now be expressed by:

$$\Delta h_{vert} = \frac{N(h_m - h_o)}{k_1}\left[1 - \frac{\frac{1}{2}k_1(h_m - h_o)}{k_2H_2 + k_1(h_m - h_o)}\right] \qquad (30)$$

Analogous to the case of the homogeneous aquifer a simplification of the expression for Δh_{vert} can be accepted by neglecting the second term between the brackets, giving:

$$\Delta h_{vert} = \frac{N}{k_1}(h_m - h_o) \qquad (31)$$

The vertical flow can be incorporated in the Hooghoudt formula by adding Δh_{vert} to the potential difference used in Eq.29. The result is an expression similar to Eq.22:

$$\frac{N}{k_1 - N} = \frac{U}{k_1 - N} = \frac{8\frac{k_2}{k_1}d(h_m - h_o) + 4(h_m - h_o)^2}{L^2} \qquad (32)$$

The expression which has to be substituted for d can be found again in 13, but now with proper subscripts:

$$\frac{d}{H_2} = \frac{L}{L + 8k_2H_2\Omega_0} \qquad (33)$$

Substitution of Eq. 33 in 32 and re-arranging results in:

$$\frac{NL^2}{4(k_1 - N)(h_m - h_o)^2} - 1 = \frac{2k_2H_2L}{k_1(h_m - h_o)(L + 8k_2H_2\Omega_0)} \qquad (34)$$

Principally there is no difference between Eqs.14 and 34. Therefore Eq.19 and Fig.5 can be used again for a solution of L. For λ, β_1 and γ the following expressions, slightly different from those used before, hold:

$$\frac{L}{8k_2H_2\Omega_0} = \lambda \qquad (35)$$

$$\frac{k_1(h_m-h_o)}{2k_2H_2} = \beta_1 \qquad (36)$$

$$2k_1\Omega_0 \sqrt{\frac{N}{k_1-N}} = \gamma \qquad (37)$$

5. Simplified expression for the drain spacing

Some attempts have been made to obtain simple expressions for the drain spacing. The first attempt was made by adding an empirical coefficient to the last term of equation 7:

$$h_m-h_o = \frac{NL^2}{8k_2H_2 + 4k_1(h_m-h_o)} + \beta_2 NL\Omega_0 \qquad (38)$$

with

$$\beta_2 = \frac{k_2H_2}{k_2H_2 + \frac{1}{2}k_1(h_m-h_o)} = \frac{1}{1+\beta_1} \qquad (39)$$

The introduction of the coefficient β_2 was done with the intention to avoid the use of more complicated expressions for Ω. This results in an equation of the second degree in L:

$$h_m-h_o = N\frac{L^2 + 8k_2H_2\Omega_0\ L}{8k_2H_2 + 4k_1h} \qquad (40)$$

However, Eq. 40 is only sufficiently accurate if $\gamma \simeq 2k_1L_0\sqrt{N/k_1} < 0.1$. This is enough reason to reject Eq.40.

By solving L^2 from Eq. 7, adding a second degree term in h similar to Eq.1 and using the radial flow resistance $\beta_2\Omega_0$ similar to Eq.38, a simplified expression of satisfactory accuracy has been obtained, namely:

$$NL^2 = 4k_1(h_m-h_o)^2 + N\left[-4\beta_2k_2H_2\Omega_0 + \sqrt{(4\beta_2k_2H_2\Omega_0)^2 + \frac{8k_2H_2(h_m-h_o)}{N}}\right]^2 \qquad (41)$$

Introduction of similar parameters λ, β_1 and γ as before, however with $\sqrt{N/k_1}$ instead of $\sqrt{N/(k_1-N)}$, permits a shorter writing of the last formula:

$$\lambda^2 = \frac{\beta_1{}^2}{\gamma^2} + \frac{1}{4(1+\beta_1)^2}\left[-1+\sqrt{1+\frac{4\beta_1(1+\beta_1)}{\gamma^2}}\right]^2 \tag{42}$$

$$\frac{L}{8k_2H_2\Omega_0} = \lambda \tag{43}$$

$$\frac{k_1(h_m-h_o)}{2k_2H_2} = \beta_1 \tag{44}$$

$$2\ k_1\Omega_0\sqrt{\frac{N}{k_1}} = \gamma \tag{45}$$

Eqs. 41 and 42 may give some advantage when a calculation of λ is required and no nomograph is available. The rather small differences in γ-values according to Eqs.19 and 42 are shown by the dotted lines in Fig.5. Errors of more than 5% will only occur when $\lambda < 1$ and $\gamma > 0.3$.

6. The drain spacing for a three-layer aquifer

Because Eq. 34 is applicable to two-layer aquifers, with the restriction that the interface between the two layers has to be of the same depth as the level of the open water, it seems profitable for practical application to also investigate the case of an interface below the bottom of the drainage channels.

This can be done by immediately passing over to the consideration of three-layer aquifers as shown in Fig.8.

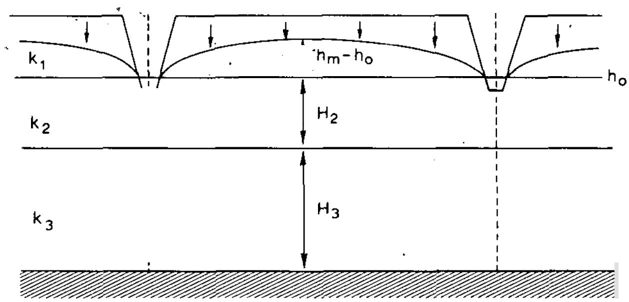

Fig.8. Parallel drains in a three-layer aquifer.

Eq.34 can be adapted to such three-layer aquifers by writing $k_2H_2 + k_3H_3$ instead of k_2H_2. Moreover a resistance Ω_{20} will be introduced for the radial flow in the two layers below the open water level. This means that hardly anything new will be met in the following Eqs.46 to 49.

$$\frac{NL^2}{4(k_1-N)(h_m-h_o)^2} - 1 = \frac{2(k_2H_2 + k_3H_3)L}{k_1(h_m-h_o)\{L + 8(k_2H_2 + k_3H_3)\Omega_{20}\}} \tag{46}$$

Eq.19 is again valid when the following expressions for λ, β_1 and γ are used:

$$\lambda = \frac{L}{8(k_2H_2 + k_3H_3)\ \Omega_{20}} \tag{47}$$

$$\beta_1 = \frac{k_1(h_m-h_o)}{2(k_2H_2 + k_3H_3)} \tag{48}$$

$$\gamma = 2\ k_1\Omega_{20}\ \sqrt{\frac{N}{k_1-N}} \tag{49}$$

For the application of the preceding equations the determination of the transmissivity for horizontal flow in each of the three layers is required. This will give no special difficulties compared with the more simple cases. So there remains only the determination of the radial flow resistance Ω_{20} as a new problem, asking for a separate treatment.

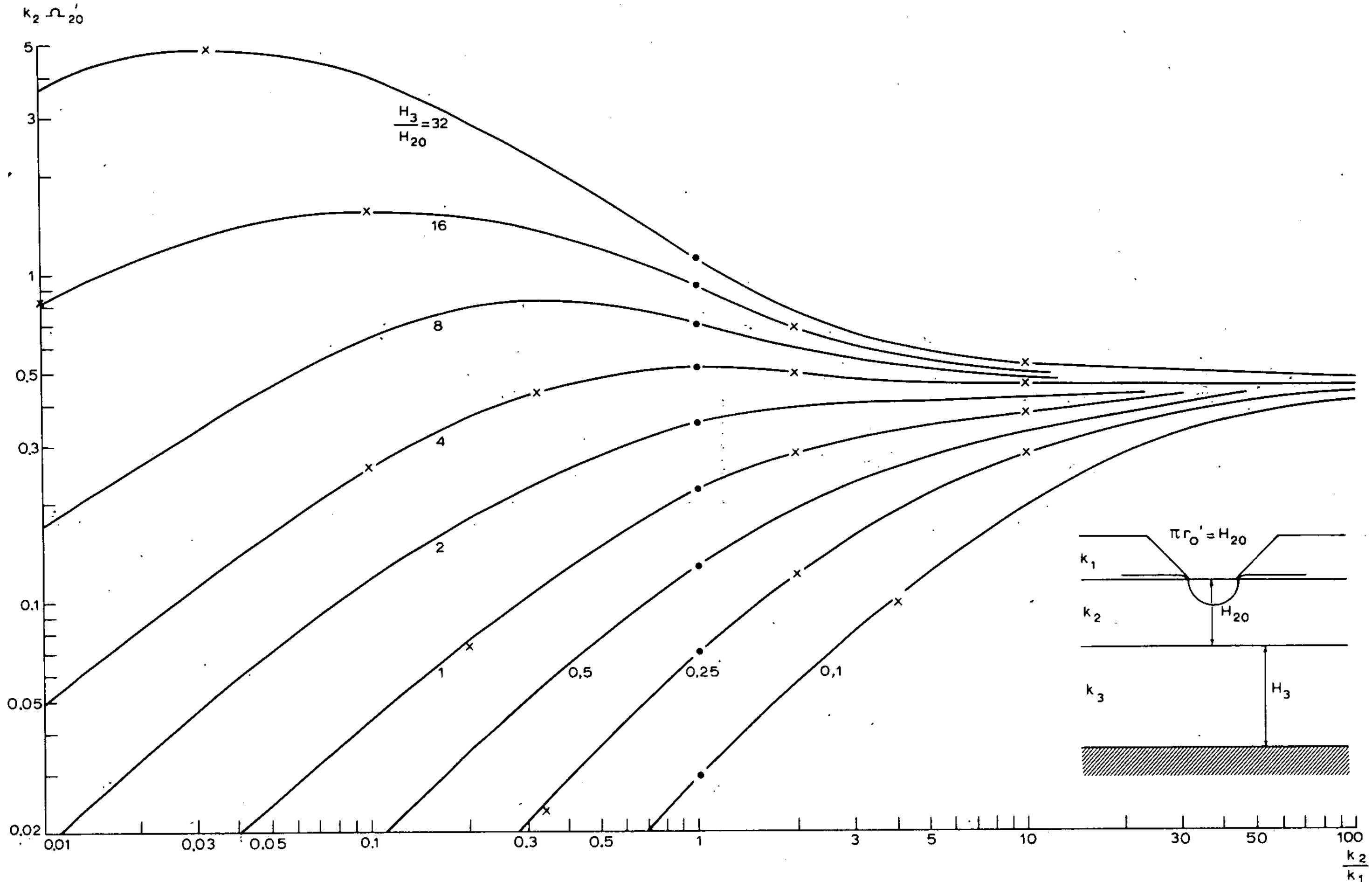

Fig. 9. Nomograph of $k_2\Omega_{20}$-values for a two-layer aquifer (Ernst, 1962, 1963). The radial flow resistance Ω_{20} can be computed by means of Eqs. 49 or 50.

Fig.9 is showing a nomograph by means of which a determination of radial flow resistance in two-layer aquifers can be obtained. In this nomograph only k_3/k_2 and H_3/H_{20} are considered as variables, while variations in size or shape of the drainage channel and the phreatic surface are neglected (ERNST, 1962, 1963).

Fig. 9 depends mainly on two assumptions:

1. use of a special value $B'_{wp} = H_{20}$ and
2. small values for the discharge q_o so that variations, comparable to what has been shown in Fig.4, are of no importance.

Only for these conditions Fig.9 is giving immediately the corresponding radial flow resistance Ω'_{20}. For arbitrary values of the wet perimeter B_{wp}, but anyhow not larger than H_{20}, the real radial flow resistance Ω_{20} can be computed by means of Eqs.50 or 51:

$$k_2\Omega_{20} = k_1\Omega'_{20} + \frac{1}{\pi} \ln \frac{H_{20}}{B_{wp}} \qquad (50)$$

$$k_2\Omega_{20} = k_2\Omega'_{20} + \frac{1}{\pi} \ln \frac{4H_{20}}{\pi B_{wp}} \qquad (51)$$

in which formulae:

Ω_{20} = radial flow resistance for a two layer aquifer with a nearly horizontal phreatic surface

H_{20} = thickness of layer with permeability k_2 between drain level and lower boundary of this layer

Which of the last two Eqs. has to be applied depends on the general shape of the channel, respectively for relatively deep and relatively shallow channels as shown in Fig.3.

7. Discussion

Use of equations with logarithms - like Eq.8 or 9 - for the determination of the radial flow resistance Ω_o may cause relatively large errors. This might be considered as a major imperfection in the presented system. In order to show to which extent this is an objection for practical use, Eqs. 7 and 8 will be applied on the drainage situation in a homogeneous aquifer (Fig. 10).

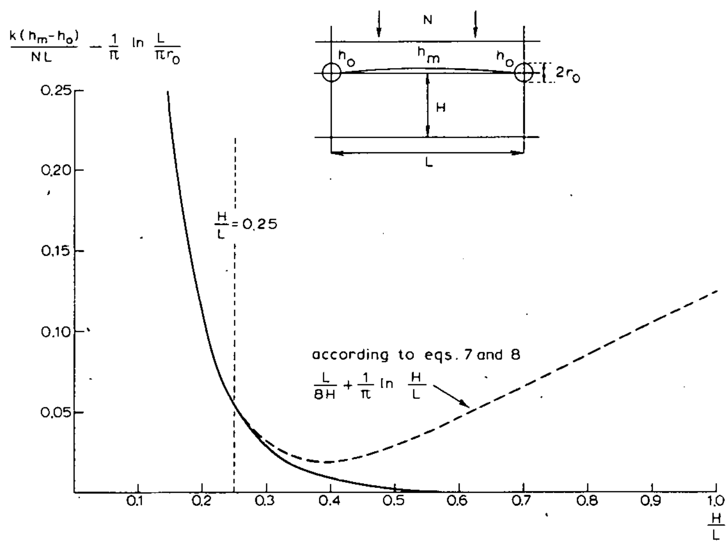

Fig. 10. Graphical representation of the hydraulic head difference $h_m - h_o$ *for the symmetric drainage of a homogeneous aquifer of constant thickness H.*

By addition of the logarithmic term to the ordinates in Fig.10, the influence of the channel size ($\pi r_o = B_{wp}$) has been eliminated. The full drawn line is giving the exact relation, while the less accurate relation according to Eqs. 7 and 8 is represented by the broken line. It is obvious that for increasing H/L above 0.25 the error is rapidly increasing, while for smaller H/L the error is completely unimportant.

For the same reason the new Eqs.14, 23, 34 and 41 should also be applied with caution when H/L > 0.25. This is even more valid for the heterogeneous aquifers dealt with in the preceding section when $(H_2 + H_3)\,L^{-1} > 0.25$ and $k_3 >> k_2$.

Some authors (HOOGHOUDT, 1940; VAN BEERS, 1965) have stated that it makes hardly any difference if the aquifer at a depth below 0.25 L is permeable or impermeable. Neglecting the deeper part of the aquifer is fairly correct, in all those cases in which the aquifer below 0.25 L will not have a very large conductivity.

In order to show the influence of the permeability of deeper layers, e.g. below a depth 0.25 L, Fig.11 has been constructed by means of available exact information (KIRKHAM, 1961; TOKSÖZ and KIRKHAM, 1971 b). This figure gives a comparison of required hydraulic head differences for two-layer aquifers, which are only different in k_2-values. The small H_1/r_o-values given in this figure, do not occur under practical conditions. When H_1/r_o = 8, it follows that $B_{wp}/L = 0.1$.

By Fig.11a it is shown that the assumption of impermeability below a depth L/4 can be rather bad, especially for small H_1/r_o-values which are not likely to exist. Even larger errors have to be expected when the second layer is neglected for values of H_1/L smaller than 0.25.

From Fig.11b it can be concluded, that in case of a complete ignorance about the deeper layers, the errors will stay between fair limits by assuming that the permeability k_1 also holds for the deeper layers below L/4.

Fig.11c shows that introduction of $k_2 = \infty$ can only be recommended in those cases that $k_2/k_1 > 3$.

A main result of Fig.11 is that it shows the relatively large influence of the H_1/r_o-values on the magnitude of the errors caused by introduction of wrong values for k_2. When relatively large drains are excluded, some ignorance about the deeper layers is much less harmful.

Finally it must be born in mind that the question about the errors caused by inaccurate values for the hydraulic conductivity of the deeper layers, has not to be confused with the applicability of the drainage formulae presented in this paper. The statement that these formulae should not be used for very thick aquifers (H/L > 0.25) has to be maintained. This restriction can hardly be weakened for relatively small drains, because even for $B_{wp}/L = 0.003$ with $H_1/L=1$, $H_2/L=1$ and $k_2/k_1=10$, it can be shown that a plain use of these formulae will result in a 25%-error.

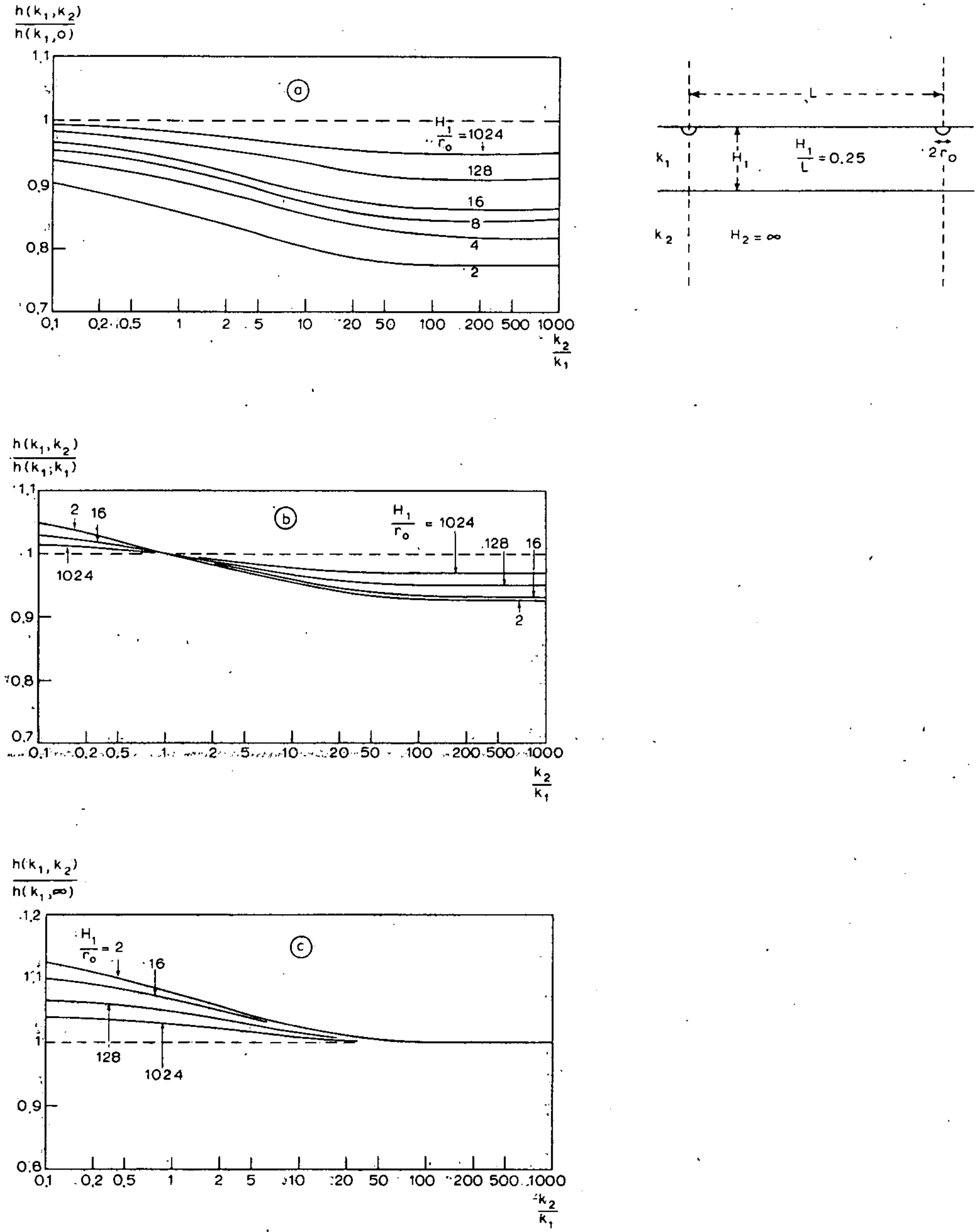

Fig. 11. The ratio between the hydraulic head differences for two symmetric drainage situations, which are only different in the k_2-value.

(a) Comparison with $k_2=0$. (b) Comparison with $k_2=k_1$.

(c) Comparison with $k_2=\infty$

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Paper 1.03

DRAINAGE CALCULATIONS IN STRATIFIED SOILS USING THE ANISOTROPIC SOIL MODEL TO SIMULATE HYDRAULIC CONDUCTIVITY CONDITIONS

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Summary

Drainage often occurs in alluvial plains, which are characterized by layered soils that have different hydraulic conductivities for horizontal and for vertical flow.

The generally applied calculation models, assuming isotropic, permeable soils are not suitable to simulate the drainage flow in layered soils. Drainage in such soils can be more suitably calculated by means of anisotropic soil models. Moreover, the anisotropy approach offers the advantage that the otherwise difficult and often arbitrary choice of the depth of the impermeable layer is of less significance. Accounting for soil anisotropy in drainage calculations is relatively easy as anisotropic models can be reduced to know isotropic calculation models by simple transformation rules.

The article further discusses the theory and application of anisotropy in drainage calculations, the determination of the horizontal and vertical hydraulic conductivity components, data on soil anisotropy from literature and collected in the field.

1. Introduction

Drainage problems usually occur in alluvial plains. Alluvial soils are characterized by macro and micro stratification. Drainage calculations are mostly based on one or two layer models assuming isotropic soil conditions. These models are not very suitable to simulate the drainage flow in stratified soil layers. They neglect anisotropy resulting from micro stratification and can only consider the effect of macro stratification on the drainage flow to a very slight extent.

It is believed that the anisotropic soil model is better suited to approach the actual conditions in a stratified soil and is, therefore, to be preferred for this exercise.

The introduction of soil anisotropy in drainage calculations for stratified soils is no new feature. Israelsen and Hansen (1960) have drawn attention to the anisotropy of alluvial soils. Lindenbergh (1963), and Boumans (1963) have also reported on this feature. Application of anisotropy in drainage, has, however, found little application so far. This paper discusses different aspects of the anisotropy model.

2. Definitions and theory of soil anisotropy

2.1 Definitions and basic principles

If the hydraulic conductivity (K) in a soil is not the same in all directions, such soil is anisotropic. For practical reasons anisotropy is limited to a different K for the horizontal flow (Kh) and for the vertical flow (Kv).

$Kh = Kv$		isotropic soil
$Kh \neq Kv$		anisotropic soil
$Kh/Kv = R$		anisotropy ratio
$1/Kv$		vertical resistivity

The theory of the flow through anositropic media has been studied by several authors. The available knowledge was reviewed by Maasland (1957) who also discusses applications and implications for the groundwater drainage.

Application of anisotropy in drainage calculations is in fact simple, as any homogeneous anisotropic soil medium can be transformed into a fictitious isotropic medium by easy rules. Transformation makes it possible to apply the available drainage formulas which take into account two dimensional flow for isotropic soil as well as for anisotropic soil. For formulas based on one dimensional horizontal flow only, e.g. the Donnan equation (1964), transformation has no sense and does not affect results.

2.2 Transformation of anisotropic parameters into isotropic parameters

Transformation is done by replacing Kh and Kv by their geometric average and by extending the vertical dimensions of the flow medium by a factor $R^{\frac{1}{2}}$ or by reducing the horizontal dimensions by the factor $R^{-\frac{1}{2}}$. Extension of vertical dimensions, the most practical way, has been further worked out.

If K_{1h}, K_{1v}, K_{2h}, K_{2v}, R_1, R_2, D_1, D_2, S, U, h and q are the parameters of the anisotropic two-layer model, the parameters of the fictitious isotropic two-layer model are as shown in Fig.1.

Transformed isotropic		Factual anisotropic
K_1^1	=	$K_{1h}^{\frac{1}{2}} \cdot K_{1v}^{\frac{1}{2}} = K_{1h}R_1^{-\frac{1}{2}}$
K_2^1	=	$K_{2h}^{\frac{1}{2}} \cdot K_{2v}^{\frac{1}{2}} = K_{2h}R_2^{-\frac{1}{2}}$
D_1'	=	$D_1\ R_1^{\frac{1}{2}}$
D_2^1	=	$D_2\ R_2^{\frac{1}{2}}$
U^1 pipe	=	$\pi\ (r + rR^{\frac{1}{2}})$
U^1 general	=	1 x horizontal + $R^{\frac{1}{2}}$ x vertical dimension of drain

Horizontal dimensions, hydraulic head and discharge remain unchanged, thus

$$S^1 = S,\ h^1 = h,\ q^1 = q.$$

The rules are applicable to the one-layer model by taking $D_2 = 0$.

In certain equations the same symbol (h) is used for the hydraulic head and for the thickness of the flow profile above the drain level. For transformation according to the method given above, both components have to be separated and treated differently. If separation is complicated, the trans-

FIG. 1ᵃ TWO LAYER MODEL FOR ANISOTROPIC SOIL

FIG. 1ᵇ TRANSFORMATION RULES

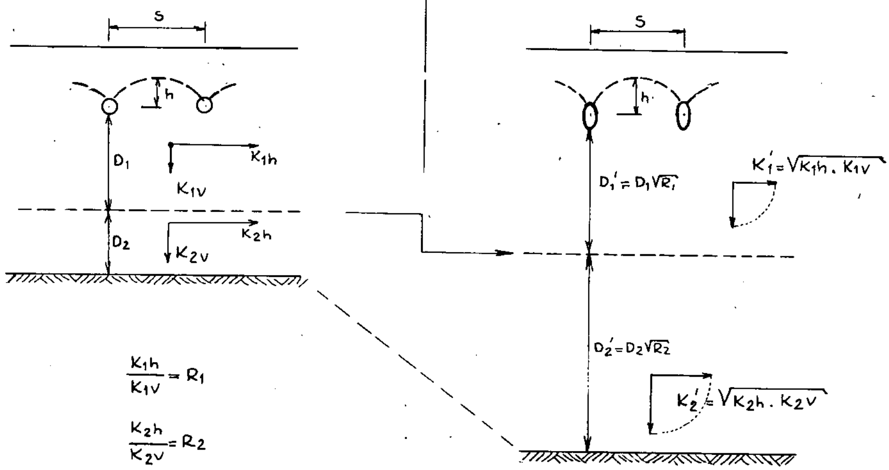

FIG. 1ᶜ TRANSFORMATION OF WET PERIMETER U

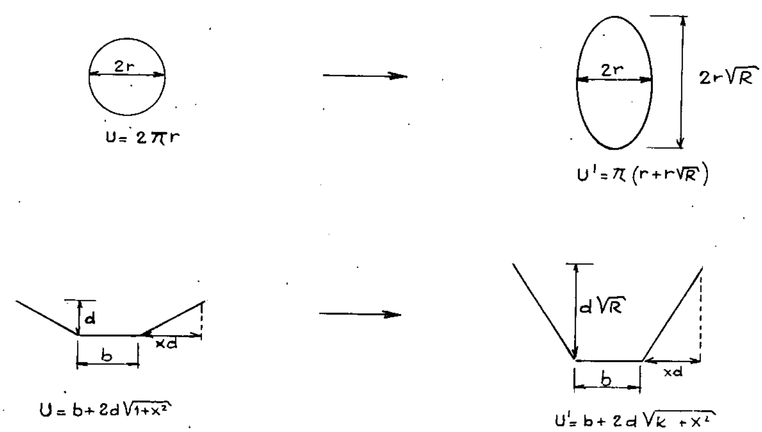

FIG. 2ᵃ ONE LAYER ISOTROPIC MODEL

FIG. 2ᵇ TWO LAYER ISOTROPIC MODEL

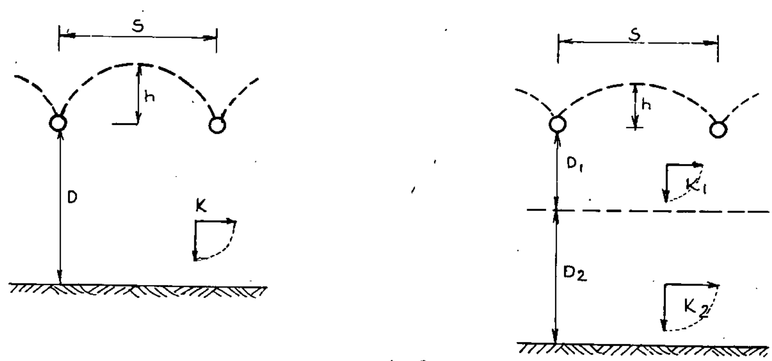

FIG. 1 + 2

formation method, in which the horizontal dimension are reduced and the vertical ones remain unchanged, is to be preferred.

2.3 Depth of relevant soil profile

The relevant soil depth (RD) is the depth below drain level that is of importance to the drainage flow if no impermeable layer occurs. Soil permeability investigations should go as deep as the RD unless an impermeable layer is met. If no impermeable layer is found within the RD, drainage formulas assuming $D = \infty$ may be used.

For practical use the RD can be taken to be equal to 1/4 of the drain spacing in isotropic soil and equal to $\frac{1}{4}R^{-\frac{1}{2}}$ times the drain spacing in anisotropic soil.

The effect of anisotropy on soil depth relevant for drainage, and thus for the soil investigations, is demonstrated in Table 1.

TABLE 1. Relevant depth of soil profile in metres below drain level

$\frac{K_h}{K_v} = R$	Drain spacing in metres			
	20	50	100	200
1	5	12.5	25	50
4	2.5	6.2	12.5	25
9	1.7	4.2	8.3	16.6
16	1.25	3.1	6.2	12.5
25	1	2.5	5	10

3. Stratified alluvial soils

Drainage problems mainly occur in alluvial plains. These plains are characterized by stratified deposits due to the sedimentation processes.

Distinction is to be made between macro and micro stratification.Macrostratification refers to the succession of layers of different textures and physical properties in the profile, micro stratification to the platy or laminated structure of the separate layers. Both macro and micro stratification can be observed in soil pits and borings.

Alluvial soils are not only heterogeneous in vertical direction, they also show a great variation horizontally. Soil profiles vary from place to place. Soil strata are seldom found continuously over a large area. They often occur in lenses of a limited extent or are intersected by other deposits.

4. The drainage model for isotropic soil

The actual flow of groundwater to drains in the fields is very complicated. For practical application, soil and flow conditions have to be reduced to simplified simulation models. The models commonly used for drainage design purposes are:

- the one-layer isotropic soil model of Fig.2, for which solutions are available for steady state and non-steady state flow assumptions, such as the Hooghoudt and Ernst formulas for steady state and the modified[1] Glover-Dumm and Krayenhoff van de Leur-Maasland formulas for non-steady state conditions;
- the two-layer isotropic soil model of Fig.1a for which steady state solutions by Ernst and Toksöz-Kirkham are available.

The isotropic models are not suited to simulate flow conditions in stratified alluvial soils for the following reasons:

- soil anisotropy due to micro stratification is neglected;
- only two permeable layers can be accounted for although often more than two occur in the soil depths relevant for drainage;
- the depth of the impermeable layer, an essential parameter in this model, is difficult to assess. Impermeability in the sense of the model is a complex notion depending on the relative permeability of a layer but also on its thickness and depth in the profile and on the extension and variation of these characteristics in horizontal direction.

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1 *"modified" refers to the replacement of depth D by the equivalent depth d of Hooghoudt to account for radial flow*

5. The drainage model for anisotropic soil

In the anisotropic one- and two-layer drainage model, the layers are assumed to be homogeneous. These models are better suited to simulate the hydraulic conductivity conditions in stratified soil.

Anisotropy due to micro-stratification can be taken into account, the effect of the complex macro-stratification can be simulated as apparent anisotropy, and the depth of the impermeable layer is less critical.

The different components of the K in the anisotropy model of a stratified soil (see Fig.3) are:

- the Kh and Kv of the separate homogeneous anisotropic soil layers: Knh and Knv;
- the average Kh and Kv of the stratified profile.

$$\bar{Kh} = \frac{\text{sum (Knh . Dn)}}{\text{sum Dn}} \qquad \bar{Kv} = \frac{\text{sum Dn}}{\text{sum } \frac{\text{Dn}}{\text{Knv}}}$$

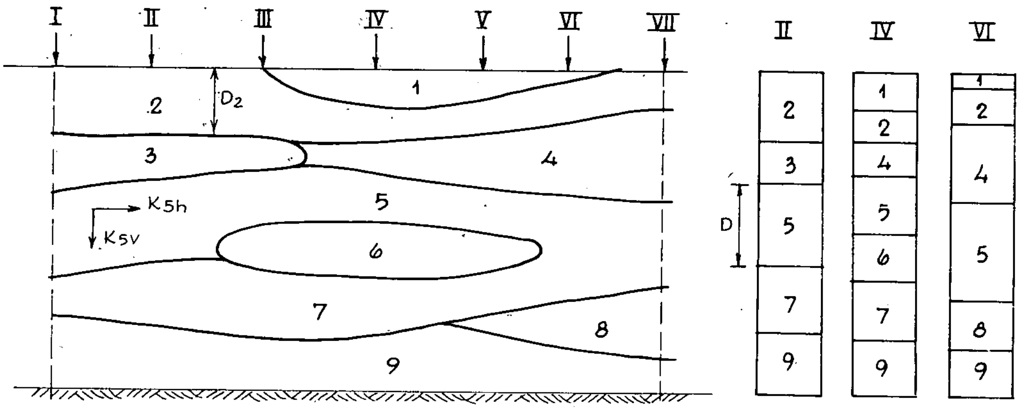
- the weighed average $\bar{Kh}$ or $\bar{Kv}$ of a section or area taking into account the variability in horizontal direction;
- the average depth of the impermeable layer if such layer occurs within the relevant soil depth.

6. Determination of the hydraulic conductivity components

6.1 Undisturbed samples

Kh and Kv of the separate soil layers can be measured in undisturbed soil samples. Average Kh and Kv values of the profile and of areas are further found by calculation. Application of the method is limited by the difficulty of obtaining a large number of undisturbed subsoil samples and by the large random error of the measurements due to the small sampling volume.

H.C. RELATIONSHIPS



1, 2, 3 ---- = n = NUMBER OF SOIL LAYER

I, II, III ----- = m = NUMBER OF SOIL BORING (PROFILES)

D_n^{II} = THICKNESS OF SOIL LAYER n IN PROFILE II

K_{nh} = HORIZONTAL H.C. SOIL LAYER n

K_{nv} = VERTICAL H.C. SOIL LAYER n

$$\left.\begin{aligned} \overline{K_h I} &= \frac{\sum K_{hn} D_n^I}{\sum D_n^I} \\ \overline{K_v I} &= \frac{\sum D_n^I}{\sum \frac{D_n^I}{K_{vn}}} \end{aligned}\right\} \text{AVERAGE HORIZONTAL AND VERTICAL H.C. OF SOIL PROFILE AT BORING I}$$

$$\left.\begin{aligned} \overline{K_h} &= \frac{1}{m} \sum_{I}^{m} \overline{K_h} \\ \overline{K_v} &= \frac{1}{m} \sum_{I}^{m} \overline{K_v} \end{aligned}\right\} \text{AVERAGE HORIZONTAL AND VERTICAL H.C OF A SECTION OR AREA}$$

FIG. 3

6.2 Pumping tests

Pumping tests can provide data on the average Kh and Kv of a relatively large soil volume.

- If the well fully penetrates into the aquifer, the test provides the Kh provided that the depth of the aquifer is known.
- If the well partially penetrates into a non-confined aquifer, the test may provide data on the average horizontal and vertical H.C. components.
- A well test in a semi-confined aquifer supplies data on the average vertical hydraulic conductivity of the semi-permeable upper strata provided that the total depth of these strata is known.

6.3 The augerhole method

The augerhole method according to Ernst and Kirkham can be used in field measurements of the average horizontal permeability of the soil below the groundwater table. As the methods have been developed for homogeneous isotropic soil conditions, the actual Kh is in fact higher than the measured K, Kh = CK. The correction factor C has been analysed by Maasland (1957). For practical application it can mostly be neglected as its effect on the drain spacing, being proportional to the fourth root of C, is very slight.

6.4 Piezometer data

Hydraulic head differences observed in piezometers placed on top of, and below permeable soil strata supply information on the Kv of separate, or combinations of, soil layers. If the corresponding upward or downward groundwater flow rate is known, Kv can be calculated.

6.5 Indirect methods

Indirect methods such as evaluation of horizontal and vertical permeability on the basis of texture or by visual appraisal of soil structure and micro-stratification are very useful, especially if they serve to interpolate or extrapolate other data.

6.6 Measurements on existing drains

Analyses of the discharge - hydraulic head curve (q - h curve) of an existing drain assuming isotropic soil, result in an apparent $K' = Kh^{\frac{1}{2}}\ Kv^{\frac{1}{2}}$. If Kh is known from pumping tests, augerhole measurements, or otherwise, Kv can be calculated.

7. Data on soil anisotropy

7.1 Individual soil strata

Numerous laboratory analysis have been made of the vertical and horizontal components of hydraulic conductivity of soil samples. Some data up to 1957 have been reviewed by Maasland (1957). It was found that for all soils, except for some sands, Kh exceeds Kv, the ratio Kh/Kv ranging from 2 to 40.

Some data for The Netherlands have been reported by Boumans (1963) for very permeable sandy subsoils and by Kessler et al. for loamy, laminated tidal flat deposits. The Kh/Kv ratios for sands ranged from 1.1 to 3.4 with an average of 1.9. The Kh/Kv for the four tidal flat samples ranged from 1 to 6 with an average of 4. The anisotropy due to micro-stratification is evident, but the degree to which it occurred, proved very variable.

7.2 Stratified profiles

No inventory has yet been made of available data on the apparent anisotropy of soil profiles and larger soil bodies. Some data collected by the author in Iraq, Iran and Pakistan will be given.

Iraq

Haddad, Elenbaidy and Hawa have reported on the results of pumping tests in the Fudhailiyah fields near Baghdad.

The profile consists of a semi-permeable aquitard of 14 m thickness, overlying a highly permeable aquifer. Pumping tests in the aquifer gave as result a vertical resistance C = D/Kv of 600 days for the aquitard, corresponding with $Kv = 14/600 = 0.023$ m/day. From shallow pump tests in the aquitard, a Kh of 0.71 m/day was calculated. The anisotropy factor $R = 0.71/0.023 = 31$.

Measurements of discharge and hydraulic head in two parallel drain ditches, spaced 1,100 m, in the Diwaniyah-Kifl area in Iraq (unpublished NEDECO report) gave as result an apparent isotropic

$$K' = \sqrt{KhKv} \text{ of } 0.41 \text{ m/day.}$$

Auger hole measurements in 5 to 10 m deep holes gave for Kh an average of 2.0 m/day. Thus Kv becomes 0.084 m/day and R = 24.

Permeability tests in 5 to 10 m auger holes made for drainage investigations in the Ammarah area in Iraq gave for Kh the average value of 1.4 m. The profiles were strongly stratified.

The average Kv calculated for the average profile, using laboratory values for the Kv of the separate soil layers, was 0.06 m/d and the corresponding R = 23.

Iran

Measurements of discharge and hydraulic head in 40 m spaced drains in the Dooroodsan scheme in the Khor River valley gave an apparent hydraulic conductivity of 0.31 m/day. Augerhole measurements in the same area gave for Kh the average value of 2.2 m/day. From these data follows Kv = 0.044 m/day and R = 50.

Pakistan

Hydrological investigations, including many pumping tests, have been carried out in the Lower Indus Plain by Hunting-MacDonald Consulting Engineers (1965 and 1971).

The hydrological profile consists of a highly permeable aquifer overlain by less permeable upper strata. The hydraulic conductivities reported are shown in Table 2.

TABLE 2. Hydraulic conductivity Indus Basin

	Total area Lower Indus Project		Khairpur area	
	Range	Typical value	Range	Typical value
Horizontal permeability of upper soil: K_h (m/day)	0.015-30	1.6	0.9-3.0	1.6
Anisotropy ratio of upper soil $R = K_h/K_v$	1-25	7	4-14	7
D_1 (see Fig. 1)	0.3-30	16	0.3-3.5	1.8
Horizontal permeability of lower soil: K_h (m/day)	0.3-30	15	27.5-39.5	33.5
Anisotropy ratio of lower soil: $R = K_h/K_v$	10-100	25	13-70	30

8. Example of drain spacing calculation

Alluvial plain of which the subsoils are very stratified, and show alternatively sandy, silt loam, clay loam and silty clay layers. Hydraulic conductivity was measured in 5 and 10 m auger holes and in undisturbed samples. The average Kh was 1.5 m/day. Kh was found to be correlated with texture.

Texture	Geometric average Kh
sand, loamy sand, sandy loam	3.2 m/day
silt loam, loam	2.2
silty clay loam, clay loam	1.4
silty clay	0.8

The anisotropy ratio was evaluated to be in the order of 25 to 50. No impermeable layer was identified in the 10 m auger holes. The one-layer steady state calculation model was adopted. Drain depth was 2 m to 2.25 m, the required average depth of the watertable during discharge periods 1 m, the corresponding hydraulic head (h) = 1.0 and 1.25 m, and the steady state design discharge 4 mm/day.

The calculations for three values of Kh, two values for R and h = 1.0 m, are summarized in Table 2. For comparison's sake, also the required spacing has been calculated for R = 1 that is if anisotropy is neglected. Drain spacing has been calculated with the aid of the Hooghoudt formula, neglecting the flow above the drain level (see Table 3).

TABLE 3. Calculations of drain spacing

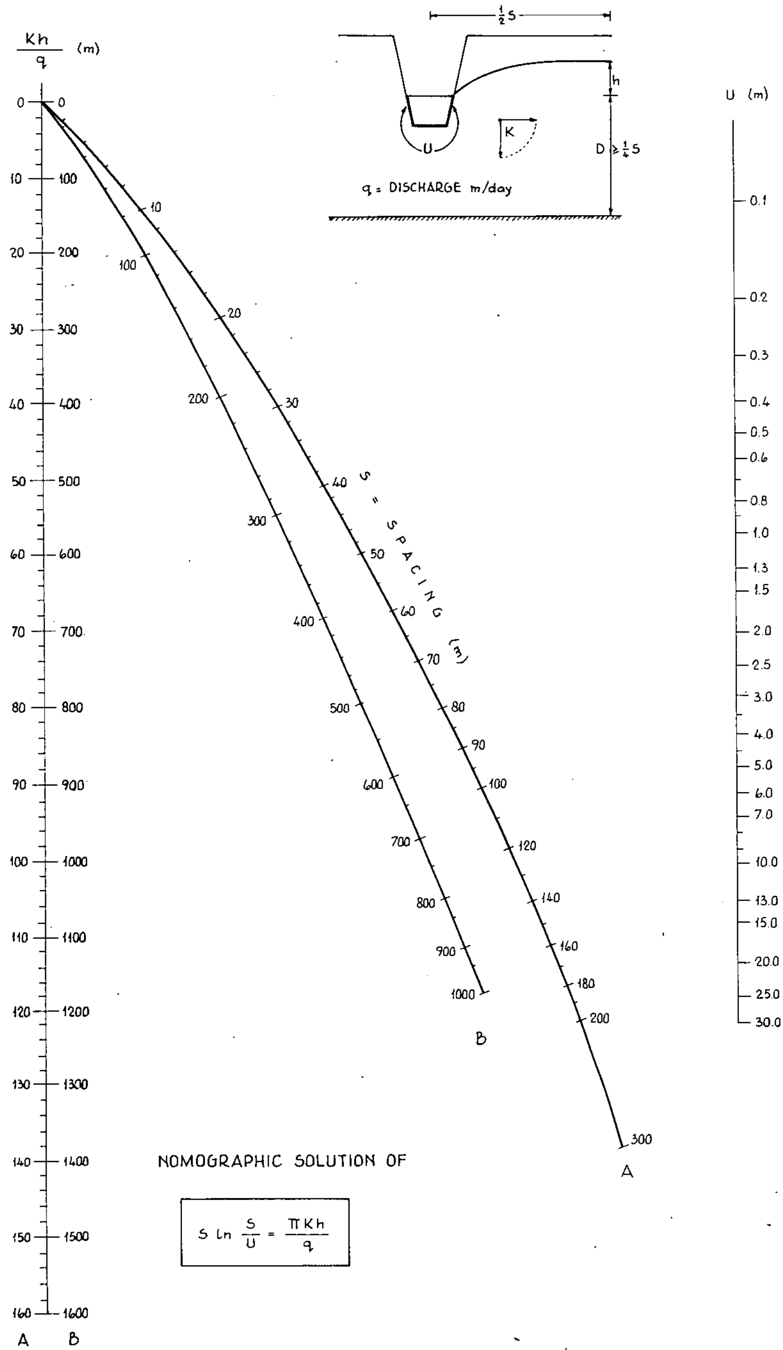
Kh, m/day		0.75			1.5			0.3	
U, m		0.6			0.6			0.6	
D, m		10			10			10	
Q, m/day		0.004			0.004			0.004	
R	50	25	1	50	25	1	50	25	1
Reduced values									
K^1, m/day	0.11	0.15	0.75	0.21	0.30	1.5	0.42	0.60	3.0
D^1, m	70	50	10	70	50	10	70	50	10
U^1, m	3.0	2.2	0.6	3.0	2.2	0.6	3.0	2.2	0.6
S = spacing, m	36	42	100	57	68	148	94	120	215
Relevant depth below drain level	1.3	2.2	25	2	3.4	37	3.4	6	54

Discussion

1) The calculations for Kh = 1.5 have resulted in an average drain spacing of 60 m, if anisotropy is taken into account. This spacing has been recommended. If anisotropy had been neglected, the average spacing would have become 150 m. The 60 m spacing corresponds well with the experience on a nearby estate situated on similar soils. Here, the drains of 2 m depth were initially spaced at 100 m, which appeared to be insufficient. The spacing was, therefore, reduced to 50 m.

2) No impermeable layer was found within the relevant soil depth for the anisotropy models. This will occur very often for anisotropy ratios of 16 and over. This implies that: (a) the use of the one-layer model is usually justified; (b) the calculation may be made with the aid of the formula for $D > \frac{1}{4}$ S:

$S \ln \frac{S}{U} = \frac{\pi Kh}{q}$ for which a simple nomogram is presented below.



Kh/q (m)
U (m)
½ S
h
D ≥ ¼ S
K
U
q = DISCHARGE m/day
S = SPACING (m)
A
B
NOMOGRAPHIC SOLUTION OF
$S \ln \frac{S}{U} = \frac{\pi K h}{q}$

FIG. 4

9. Glossary and symbols

Soil anisotropy: Soil for which the hydraulic conductivity is not the same in all directions. In practice difference is made between the hydraulic conductivity for horizontal and vertical flow.

Relevant soil depth RD: Depth below drain level of the soil profile that is of importance to the groundwater flow to the drains. RD isotropic soil is 0.25 S, RD anisotropic soil is 0.25 $SR^{-0.5}$.

HC or K	= hydraulic conductivity for saturated flow
Kv	= HC for vertical flow
Kh	= HC for horizontal flow
R = Kh/Kv	= anisotropy ratio
Knh	= Kh of n^{th} soil layer
Knv	= Kv of n^{th} soil layer
Dn	= thickness of n^{th} soil layer
$\overline{Kh}$, $\overline{Kv}$	= average Kh or Kv of stratified soil profile
S	= drain spacing
U	= effective wet perimeter of drain
h	= hydraulic head
q	= discharge rate
K^{l}, D^{l}, U^{l}	= values of K, D, U after transformation

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Paper 1.04

THE HYDRAULIC CONDUCTIVITY IN HETEROGENEOUS AND ANISOTROPIC MEDIA AND ITS ESTIMATION IN SITU

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Summary

The equations of groundwater flow used for land drainage calculations are generally derived with the assumption that the soil is homogeneous and isotropic with respect to hydraulic conductivity, or at most homogeneous and anisotropic.

Introducing the "equivalent horizontal hydraulic conductivity", one of the authors has shown (WOLSACK, 1978) that the theory applied for heterogeneous soils too.

The present paper, after recalling the definition of the equivalent horizontal hydraulic conductivity, analyzes the well and piezometer method (in the case of well attaining the impervious layer), which provides an "in situ" estimation of it.

Introduction

Estimating the soil hydraulic conductivity is a major problem of predrainage studies. French experience has shown that field methods must always be preferred to laboratory methods, which generally lead to dispersed results.

Moreover, the methods which proved to give best agreement with data obtained after drainage are the pumping test methods. The so-called "well and piezometer method" is one of them, which needs use of only one piezometer for estimating hydraulic conductivity. This method has been completely described in GUYON (1971) in the homogeneous case. This paper shows how it can be adapted to a more general heterogeneous situation.

Notations

d	water level in the well	(L)
F	horizontal flow potential	(L^2T^{-1})
h	free surface elevation	(L)
i	infiltration rate (positive downwards)	(LT^{-1})
K, K_h, K_v	isotropic, horizontal and vertical hydraulic conductivity	(LT^{-1})
$\bar{K}_h(z)$	equivalent horizontal hydraulic conductivity	(LT^{-1})
q_x, q_y, q_z	components of the apparent velocity	(LT^{-1})
q_r	radial component of the apparent velocity	(LT^{-1})
Q	pumping discharge	(L^3T^{-1})
$r=(x^2+y^2)^{1/2}$	radial horizontal space coordinate	(L)
r_A	distance from the well axis to the piezometer	(L)
r_p	well radius	(L)
S_y	drainable porosity	(1)
t	time	(T)
$T(z)=\int_o^z K_h(z')dz'$	transmissivity at level z	(L^2T^{-1})
$W=\int_o^h S_y(z)dz$	drainable water height	(L)
x,y	horizontal space coordinates	(L)
z	elevation above impervious layer	(L)

The free surface problem

The general free surface problem is three-dimensional and the flow domain is bounded above by a free surface at atmospheric pressure.

It is assumed for the present study that:

- the bedrock is horizontal and impervious
- the soil is vertically heterogeneous
- any horizontal plane is an eigen-plane for anisotropy.

On any vertical, one can formulate the equations of groundwater flow as follows (Wolsack, 1976):

$$\begin{aligned}
&\phi = z && ; \; z = h \\
&\frac{\partial W}{\partial t} = - q_x \frac{\partial h}{\partial x} - q_y \frac{\partial h}{\partial y} + q_z + i && ; \; z = h \\
&\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial q_z}{\partial z} = 0 && ; \; 0 < z < h \\
&q_x = - K_h \frac{\partial \phi}{\partial x} && ; \; 0 < z < h \\
&q_y = - K_h \frac{\partial \phi}{\partial y} && ; \; 0 < z < h \\
&q_z = - K_v \frac{\partial \phi}{\partial z} && ; \; 0 < z < h \\
&q_z = 0 && ; \; z = 0
\end{aligned} \qquad (1)$$

The classical Dupuit's assumption enables reduction to a two-dimensional (horizontal) problem. For land drainage applications, this assumption is not always acceptable and it is more suitable to introduce some less strong simplifications. Guyon (1964, 1967, 1968 and 1971) has shown that the use of an auxiliary function, due to Charnyi (1951) led to a simple study in the homogeneous case (the soil being either isotropic or anisotropic). Modifying as Zaoui (1964) the definition of this auxiliary function, Wolsack (1978) has shown that the basic calculations could be reconducted in the more general situation presented above.

The horizontal flow potential

One introduces, as announced, an auxiliary function F, defined by:

$$F = \int_o^h K_h(z)\ (\phi - z)\ dz \tag{2}$$

Combining with (1), one easily obtains, after some manipulations:

$$\int_o^h q_x dz = - \frac{\partial F}{\partial x} \tag{3}$$

$$\int_o^h q_y dz = - \frac{\partial F}{\partial y}$$

$$\frac{\partial^2 F}{\partial x^2} + \frac{\partial^2 F}{\partial y^2} = \frac{\partial W}{\partial t} - i \tag{4}$$

It follows that F is, from (3), the horizontal flow potential of the problem considered here, and, from (4), the solution of some Laplace problem.

The equivalent horizontal hydraulic conductivity

One defines the equivalent horizontal hydraulic conductivity by:

$$\bar{K}_h(z) = \frac{2}{z^2} \int_o^z (z - z')K_h(z')dz' \tag{5}$$

Combining Eqs. (1), (2) and (5), one obtains, after some manipulations:

$$F = \frac{h^2}{2} \bar{K}_h(h) - \int_o^h T(z) \frac{\partial \phi}{\partial z} dz \tag{6}$$

Formula (6) is interesting for two reasons:

- the last term often appears as a corrective term (it would be zero with the Dupuit's assumption);
- in the homogeneous case, one would have obtained the same expression with K_h (constant) in place of $\bar{K}_h$.

  This property, which still holds in the drainage formulas (Wolsack, 1978), justifies the proposed terminology for $\bar{K}_h$.

The general method for obtaining drainage formulas consists in evaluating F by two means (directly from (2) or (6), and by solving (4) under the appropriate boundary conditions) and to equal the expressions derived.

An exact equation for the pumping test

Field experiments show that, with certain care (Guyon, 1971), one can obtain, for water tables typical of land drainage problems, a quasi steady state when pumping at a low discharge. The radius of influence is therefore of about several meters, which corresponds to the purpose of land drainage (Fig.1).

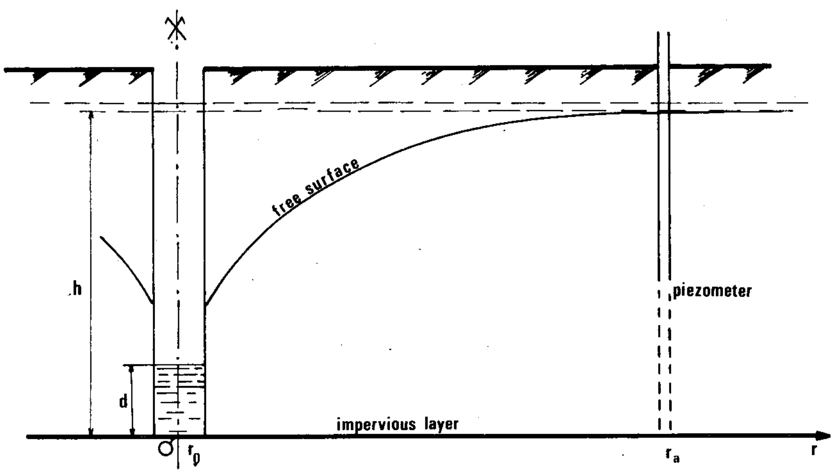


Fig.1. Fully penetrating well.

Denoting r the distance to the well axis (the well attaining the impervious layer), (4) yields, in steady state, and with no infiltration:

$$\frac{1}{r} \cdot \frac{d}{dr} \left(r \frac{dF}{dr}\right) = 0 \tag{7}$$

The boundary conditions on the well are:

$$\phi = z \qquad d < z < h\,(r_p) \tag{8}$$

$$\phi = d \qquad 0 < z < d$$

Integrating (7) with (3) and (8), one obtains, denoting Q the pumping discharge:

$$\frac{Q}{2\Pi} \ln \left(\frac{r_A}{r_p}\right) = F\,(r_A) - F\,(r_p)$$

Evaluating $F(r_A)$ from (6) and $F(r_p)$ from (2), one finally obtains:

$$\frac{Q}{\Pi} \ln \left(\frac{r_A}{r_p}\right) = h_A^2\, \bar{K}_h(h_A) - d^2\bar{K}_h(d) - 2 \int_o^{h_A} T(z)\frac{\partial\phi}{\partial z}\, dz\Big|_{r=r_A} \tag{9}$$

The last term of (9) appears as a corrective term, as smaller as r_A is greater. One developes thereafter some evaluation procedures for this term.

Evaluating the corrective term in a homogeneous and isotropic soil

In an homogeneous and isotropic soil, formula (9) yields:

$$\frac{Q}{\Pi} \ln \left(\frac{r_A}{r_p}\right) = (h_A^2 - d^2)\, K - 2\, K \int_o^{h_A} z\, \frac{\partial\phi}{\partial z}\, dz\Big|_{r=r_A} \tag{10}$$

$K \frac{\partial\phi}{\partial z}$ can be estimated as follows (from (1)):

$$0 < K \frac{\partial\phi}{\partial z}\,(r_a,z) = |q_z|\,(r_A,z) < |q_z|\,(r_A,h_A) = |q_r|\,(r_A,h_A)\,\frac{dh}{dr}\,(r_A)$$

One has, too:

$$|q_r|\,(r_A,h_A) < \frac{Q}{2\Pi r_A h_A}\;, \quad \left(\frac{\partial|q_r|}{\partial z} < 0\right)\;,$$

$$\frac{dh}{dr}\,(r_A) < \frac{Q}{2\Pi K r_A h_A}$$ *(slope of the free surface with the Dupuit's assumption)*

It follows that:

$$0 < K \frac{\partial\phi}{\partial z} (r_A, z) < \frac{1}{K} \frac{Q^2}{4\Pi \ r_A^2 h_A^2}$$

Thus, combining with (10):

$$\frac{Q}{\Pi} \ln \left(\frac{r_A}{r_p}\right) < (h_A^2 - d^2) \ K < \frac{Q}{\Pi} \ln \left(\frac{r_A}{r_p}\right) + \frac{1}{K} \frac{Q^2}{4\Pi^2 r_A}$$

Majorating the last term with the first inequality:

$$\frac{Q}{\Pi} \ln \left(\frac{r_A}{r_p}\right) < (h_A^2 - d^2) \ K < \frac{Q}{\Pi} \ln \left(\frac{r_A}{r_p}\right) \left(1 + \frac{h_A^2 - d^2}{4r_A^2 \ln^2 \left(\frac{r_A}{r_p}\right)}\right)$$

When, using (10), one neglects the last term, and the error on K is thus:

$$0 < \frac{\Delta K}{K} < \frac{h_A^2 - d^2}{4r_A^2 \ln^2 \left(\frac{r_A}{r_p}\right)} \qquad (12)$$

For example, with $h_A = 1.50$ m, $d = 0.20$ m, $r_A = 2$ m, $r_p = 0.10$ m, the relative error is less than 1.6 %.

Taking anisotropy in account

If the soil is homogeneous and anisotropic, one finds an isotropic equivalent, using the following affinity:

$$x' = x \sqrt{\frac{K_v}{K_h}} \ ; \ y' = y \sqrt{\frac{K_v}{K_h}} \ ; \ z' = z$$

One introduces too:

$$\phi' = \phi \ ; \ q'_x = q_x \sqrt{\frac{K_v}{K_h}} ; \ q'_y = q_y \sqrt{\frac{K_v}{K_h}} \ ; \ q'_z = q_z$$

One easily verifies that (1) can be written in (ϕ', q') and (x',y',z') as for an homogeneous and isotropic soil of hydraulic conductivity K_v. One can thus copy (11) with:

$$K \to K_v \; ; \; Q \to Q\,\frac{K_v}{K_h} \; ; \; r_A \to r_A\sqrt{\frac{K_v}{K_h}} \; ; \; r_p \to r_p\sqrt{\frac{K_v}{K_h}}$$

One obtains after re-arrangement:

$$\frac{Q}{\Pi}\ln\left(\frac{r_A}{r_p}\right) < (h_A^2 - d^2)\,K_h < \frac{Q}{\Pi}\ln\left(\frac{r_A}{r_p}\right)\left(1 + \frac{h_A^2 - d^2}{4r_A^2\,\ln^2\left(\frac{r_A}{r_p}\right)} \cdot \frac{K_h}{K_v}\right) \qquad (13)$$

If K_h is estimated by the approximate formula:

$$\frac{Q}{\Pi}\ln\left(\frac{r_A}{r_p}\right) \simeq (h_A^2 - d^2)\,K_h\;, \qquad (14)$$

Then the relative error becomes:

$$0 < \frac{\Delta K_h}{K_h} < \frac{h_A^2 - d^2}{4r_A^2\,\ln^2\left(\frac{r_A}{r_p}\right)} \cdot \frac{K_h}{K_v} \qquad (15)$$

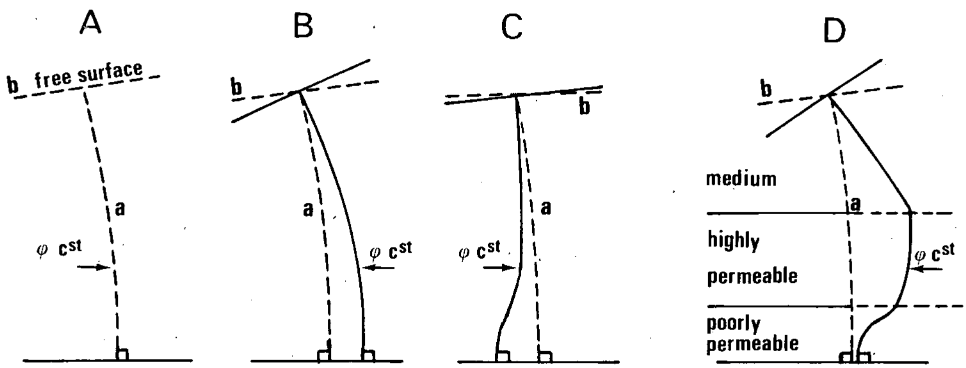
The relative error is decreasing when K_v is increasing, which is normal because, at the limit, for K_v infinite, the Dupuit's assumption (hydrostatic pressure distribution) is strictly verified.

Taking heterogeneity in account

One has now to evaluate the error appearing when neglecting the corrective term of (9), say:

$$- 2 \int_0^{h_A} T(z)\,\frac{\partial\phi}{\partial z}\Big|_{r=r_A}$$

This evaluation will be qualitative, and based on observing the evolution of the corrective term, when passing from a reference homogeneous case to the actual heterogeneous case. In a homogeneous soil, the equipotentials have the shape shown in Fig.2A.



A: $K_h(z)$ constant B: $K_h(z)$ decreasing

C: $K_h(z)$ increasing D: layered soil (example)

Fig.2. Equipotentials in an heterogeneous soil.

If the hydraulic conductivity increases with depth, the streamlines are deviated downwards and the equipotentials are less vertical, which increases the modulus of the corrective term (Fig.2B).

If the hydraulic conductivity decreases with depth, the streamlines are deviated upwards, which makes the equipotentials more vertical and the modulus of the corrective term smaller (without probably changing its sign) (Fig.2.C).

In the case of a non-monotonous variation, one obtains a combination of both effects (Fig.2D).

The well and piezometer method

According to the preceding results, one can conclude that the corrective term of (9) is generally negligible, except:

- when the anisotropy ratio $\frac{K_h}{K_v}$ is very high;
- or when there is a very permeable layer at some depth.

In both cases, which can be guessed by the soil scientist (and eventually verified with the Vergière method (Bourrier, 1965)), it is necessary to hold a particular study.

In the general case, the pumping test is conducted in order to estimate K_h with the formula (9), simplified as follows:

$$\frac{Q}{\Pi} \ln \left(\frac{r_A}{r_p}\right) \simeq h_A^2 \, \bar{K}_h \, (h_A) - d^2 \bar{K}_h \, (d)$$

First, one must remind that there is need for $\bar{K}_h(z)$ at one, two or three different levels, which leads to one, two or three successive quasi steady states.

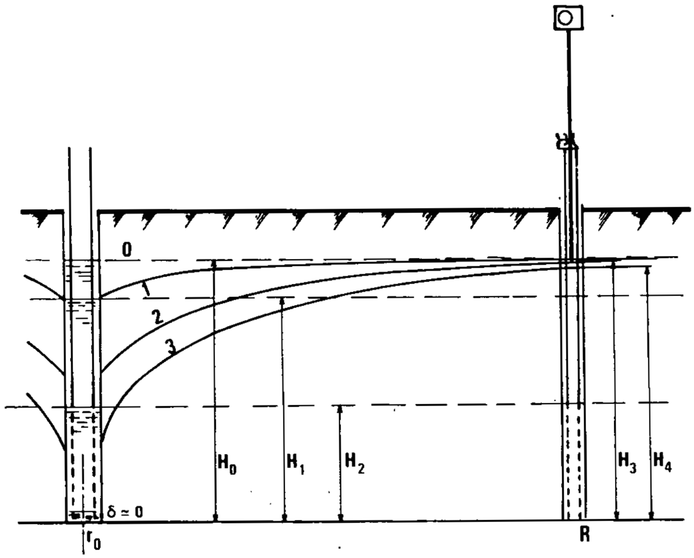


Fig. 3. Three levels in a pumping test.

Fig.3 illustrates the three-level test; the formulas providing the three values of $\bar{K}_h$ are respectively, from (16):

$$\frac{Q_1}{\Pi} \ln \left(\frac{R}{r_o}\right) \simeq H_o^2 \, \bar{K}_h \, (H_o) - H_1^2 \bar{K}_h \, (H_1) \qquad \text{(step 1)} \qquad (17)$$

$$\frac{Q_2}{\Pi} \ln \left(\frac{R}{r_o}\right) \simeq H_3^2 \, \bar{K}_h \, (H_o) - H_2^2 \bar{K}_h \, (H_2) \qquad \text{(step 2)}$$

$$\frac{Q_3}{\Pi} \ln \left(\frac{R}{r_o}\right) \simeq H_4^2 \, \bar{K}_h \, (H_o) \qquad \text{(step 3)}$$

For the two-level test, one makes steps 2 and 3 only.
For the one-level test, one makes step 3 only.

Conclusion

Estimating the soil hydraulic conductivity is a major problem of pre-drainage studies. In homogeneous cases, the "well and piezometer method" proved to be the most relevant method, according to french experience. The adaptation to heterogeneous cases demands answer to two questions:

- how to describe heterogeneous hydraulic conductivity?
- is it possible to estimate the parameters used in this description?

The first question has been given an answer in Wolsack (1978), through the introduction of the concept of "equivalent horizontal hydraulic conductivity", which proved to be well adapted to some drainage calculations.

The second question was answered in this paper, through an adaptation of the "well and piezometer method". The general procedure remains identical, but, because the equivalent horizontal hydraulic conductivity is a function of elevation, more measurements have to be done, with different water levels in the well.

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Paper 1.05

PRE-DRAINAGE RESEARCH IN LAND CONSOLIDATION AREAS

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Summary

If, in case of re-allotment, subsoil drainage is included in the plan of provisions, pre-drainage research is generally made to determine the intensity of drainage required. Apart from the objection of too a universally applied measuring and calculating technique regarding the Hooghoudt stationary flow principle, an increasing number of objections have been raised for the last years to the current method of pre-drainage research by means of the Auger hole method. These objections are generally directed at the very high costs of this labour intensive investigation and the usefulness of the result acquired. Particularly the range in permeabilities measured in a (systematic) investigation, causes problems. An intensification of the investigation usually produces no better information.

During the last few years, however, favourable experiences have been gained with research more focussed on the hydrological qualities of the profile. This approach is supported by a statistical investigation into the correlation between profile structure and the drain distance calculated by means of the Auger hole method. A strong cut down of the measuring programme is obtained by soil type classification based on hydrological qualities, by making permeability measurements in relevant profile types, and by next extrapolating the information acquired from a number of trial blocks. Dependent on the required reliability of the conclusion the number of drain distance calculations can be determined statistically. The number of permeability measurements can be reduced by half, if a classification in profile types is used and by even more, if the borings are carefully distributed over the different profile types. All this on condition that there is no relation between permeability and the depth of the impermeable layer. A further reduction of the number of permeability measurements can be reached by extending the boring depth to at least 1.50 m in the soil survey and by estimating the permeability of the different profile layers, after experience and insigh have been gained with (gauge) measurements in relevant profile layers. The extra costs of such a detailed survey are, in every respect, acceptable in view of a restriction of the later pre-drainage research. The great intensity of the profile descriptions in the soil survey (one in every 1-3 ha) makes a classification in profile types possible, in which additional measurements have been restricted to parts of the drainage plan. It should be noted that the generally stated limitation of pre-drainage research only applies to measurements,but not to borings made to determine the depth of the impermeabl layer. Especially into this matter additional investigations must be made.

This report mainly deals with the approach to and the form of pre-drainage research based on the starting-point regarding soil maps. The recommendations have always been based on the relation between profile structure and drain distance.

1. Introduction

In most re-allotments in which the installation of a sub-soil drainage system is involved, a preliminary drainage investigation is made to determine the drainage intensity required. For this investigation the Auger-hole method is used. This method comprises profile descriptions, permeability measurements and locating the depth of the impermeable layer of soil. After formulating the drainage standards, the drain distances can be calculated by means of the drainage formulas stated by Hooghoudt and Ernst.

As the area of cultivated land to be drained, increased, the objections to the method used for pre-drainage investigations also grew. The most important disadvantage was its financial aspect. Since pre-drainage investigation requires a great deal of work, particularly measuring, the current investigation method involves high preparation c.q. planning costs, caused by its average measuring intensity of one measurement (often in duplicate) in every 5-10 ha. This problem set us thinking. A statistical investigation made for that purpose, revealed to us that there is a relation between profile structure and the drain distance required, determined by means of the Auger-hole method. This means, in fact, that part of the measurements could be replaced by profile descriptions, focused on hydrological profile characteristics. If, next, permeability measurements are applied to profile types the number of measurements can be considerably cut down by extrapolation of the acquired results.

The number of measurements needed for the budget planning or for the drainage plan, can be statistically determined, dependent on the required reliability of the conclusion. It has appeared that, if a classification is used based on soil characteristics, the number of measurements can be cut down by half. A further reduction of the number of permeability measurements can be brought about by describing the soil profile till 1.50 m minimally in the soil survey, and by estimating the permeability of the different layers near and under the future drainage level (1.20 m). This estimation should be gauged previously according to permeabilities measured in profile layers which are representative in that respect. This method has already proved successful.

Another objection to a systematic and intensive use of the Auger-hole method is the reliability of the calculated drain distance. Though it has always been assumed that the right drain distance can be calculated by means of the Auger-hole method, investigations have shown a great range in permeabilities within the same soil structure. A more elaborate observation system of one in every 10 ha, to one in every 3 ha and even one in every 16 m^2, did not, however, give any better information about the exact permeability.

A general application of the Auger-hole method has formerly led to the development of quick and clear measuring and calculation techniques. The routine character that resulted from the application of these techniques has led too a universal application of the Hooghoudt and Ernst formulas for stationary state flow. This objection has been raised time and again for the last few years in the resumed discussion about agricultural discharge standards and drainage criteria. New directions are to be expected fairly soon concerning aspects such as: drainage criteria (differentiation based on trafficability and cultivation requirements, water storage and drain depth), formulas and nomogrammes (the influence of drain length and flow resistence on the drain pipe diameter).

The costs and discontent at the quality and usefulness of the acquired result have led us to take a new course for the last few years. As the present metho of pre-drainage investigation differ a great deal in every district, a unifor approach is needed using new developments and meeting the above objections. Hereto recommendations are mentioned in this report. Distinction has been mad according to the purpose of the investigation, namely whether it has been mad for budget purposes or for the drainage plan.

The Auger-hole method has been maintained as point of departure, but the accent has been shifted to soil characteristics.

2. Summary of some relevant investigation results

2.1 Relationship between profile structure and the drain distance required

If pre-drainage investigation aims at extrapolation of permeability factors by means of soil characteristics, it must be proved that there is a relation between these soil characteristics and the drain distance calculated from the Auger-hole method. This had been worked out in the re-allotment of "Slochteren". With the help of profile descriptions a division was made in soil (hydrological) units, in which the profile structure round and beneath the future drain level (about 1.20 m) has been taken full account of.

For sandy soils, a simple classification is possible based on the depth of the impermeable layer and loam percentage,so that a satisfactory correlation is obtained between these characteristics and the average drain distance. In some cases a classification according to coarseness of the sand is also considered useful. The differences in drain distance appeared to be, in most cases, statistically reliable. Variation in drain distance within the profile types formed, generally appeared to be in line with that which occurs in, homogeneous areas covering 6 ha. This surface is considered to be the smallest unit which still gets a separate drain distance.

For sand soils in the land consolidation area of "Slochteren" the results of the systematic pre-drainage investigation have been worked out as follows. A correlation between loam percentage and drain distance by different depth of the impervious layer is shown in Table 1. The drain distance has been calculated by means of the Hooghoudt formulae for steady-state flow; the K-values were measured by means of the Auger-hole method. Loam percentage is given in three classes: 1:5-10 %; 2:10-18 %; and 3:18-50 %. These classes were estimated by the profile descriptions. The loam percentage of the layers around and beneath drain level are relevant.

The difference in drain distances have been proved to be significant.

TABLE 1.

Depth of the impervious layer below surface	Loam class	Number of calculations (n)	Average drain distance ($\bar{x}$)	Standard deviation in m	Standard deviation in % of $\bar{x}$
1.00 - 1.20 m	1	15	14.1	5.5	39
1.00 - 1.20 m	2	12	13.2	5.5	42
1.00 - 1.20 m	3	4	9.8	6.4	65
1.20 - 1.60 m	1	44	22.2	8.2	37
1.20 - 1.60 m	2	53	17.2	8.1	46
1.20 - 1.60 m	3	4	13.2	2.6	20
1.60 - 2.50 m	1	114	33.1	16.3	31
1.60 - 2.50 m	2	93	24.5	7.7	32
1.60 - 2.50 m	3	9	19.6	7.1	36

For the peat-soils, in which peaty material is found at drain level, it appeared to be impossible to make such a division in profile types that it led to a correlation with the drain distance. Its variation was, however, hardly greater than the variation in drain distance in homogeneous units of peat soils of at least 6 ha. Experiences elsewhere have proved, however, that if a classification is made according to the types of peat, a differentiation in drain distance will certainly appear.

In clay soils the permeability is influenced by its lutum percentage, its degree of stratification, its stage of physical maturation and the local occurrence of a particularly permeable structure ("short clay"). A classification in profile types based on these characteristics is to be preferred for very precise drain distance calculations.

From investigations in the re-allotment area of "Harkstede" it has appeared that permeability measurements can be largely replaced by estimation. In the soil survey of the area the profile has been described till 1.50 m. In this description the permeability of each profile layer has been estimated separately after experience and insight had been gained with preliminary measurements in representative profile layers.

After a classification of the area according to relevant profile types and estimated permeability, additional permeability measurements were taken and deeper borings were made to locate the depth of the impermeable layer. A comparison of the drain distance calculated for every separate hydraulic profile type from measured permeabilities with that calculated from estimated permeabilities revealed nearly the same results.

Table 2 shows the range in K-values in two different soil types and gives the average K-value (measured and estimated) for drain distance calculations.

2.2 Approach to the number of drain distance observations

The "Slochteren" investigation also aimed at figuring the number of drain distance observations necessary for the final drainage plan only, or for budgeting. To this end formulas have been deduced, which have been graphically worked out in Figures 1 and 2.

TABLE 2.

Soil type	Depth of the impervious layer	K		Range and frequency of the K-value in m/day								Medium-value of K (m/day)
				≤0.10	0.10 to 0.20	0.20 to 0.40	0.40 to 0.60	0.60 to 0.80	0.80 to 1.20	1.20 to 1.80	≥1.80	
peaty soil with shallow podzol	≤ 1.20 m below surface	above	M*	20	9	13	2	5	2	1	-	0.17 m/d
			E	50	136	57	17	3	4	5	-	0.16 m/d
sand soil with podzol characteristics	1.80 m	above	M	5	4	6	5	2	1	1	-	0.30 m/d
		below	M	2	4	9	3	1	-	1	4	0.35 m/d
		above	E	20	132	169	80	18	6	10	5	0.28 m/d
		below	E	10	69	172	107	48	19	9	6	0.36 m/d

* *M = measured*

E = estimated

Though from these figures the number of permeability measurements for every separate profile type can be calculated, it may not be concluded that, as the number of separate profile types increases, the total number of measurements to be taken will increase in proportion. Usually the drain distance of a profile type can be deduced from another profile type. Soil types with, for instance, the same loam percentage differ, in a hydrological respect, exclusively in the depth of the impermeable layer.

Permeability measurements are, therefore, only wanted for soils with a different loam percentage. Thus the total number of measurements can be considerably cut down. However, there must be no relation between the permeability of the soil and the depth of the impermeable layer.

Figures 1 and 2 are based on the formula:

$$V = \frac{tn-1}{\sqrt{n}} \cdot \frac{Sx}{\bar{x}} \cdot 100$$

in which

V = the maximum deviation (in %) allowed between the calculated and the factual average drain distance

Sx = standard deviation from the original classification

$\bar{x}$ = the average drain distance in every profile type

n = number of observations

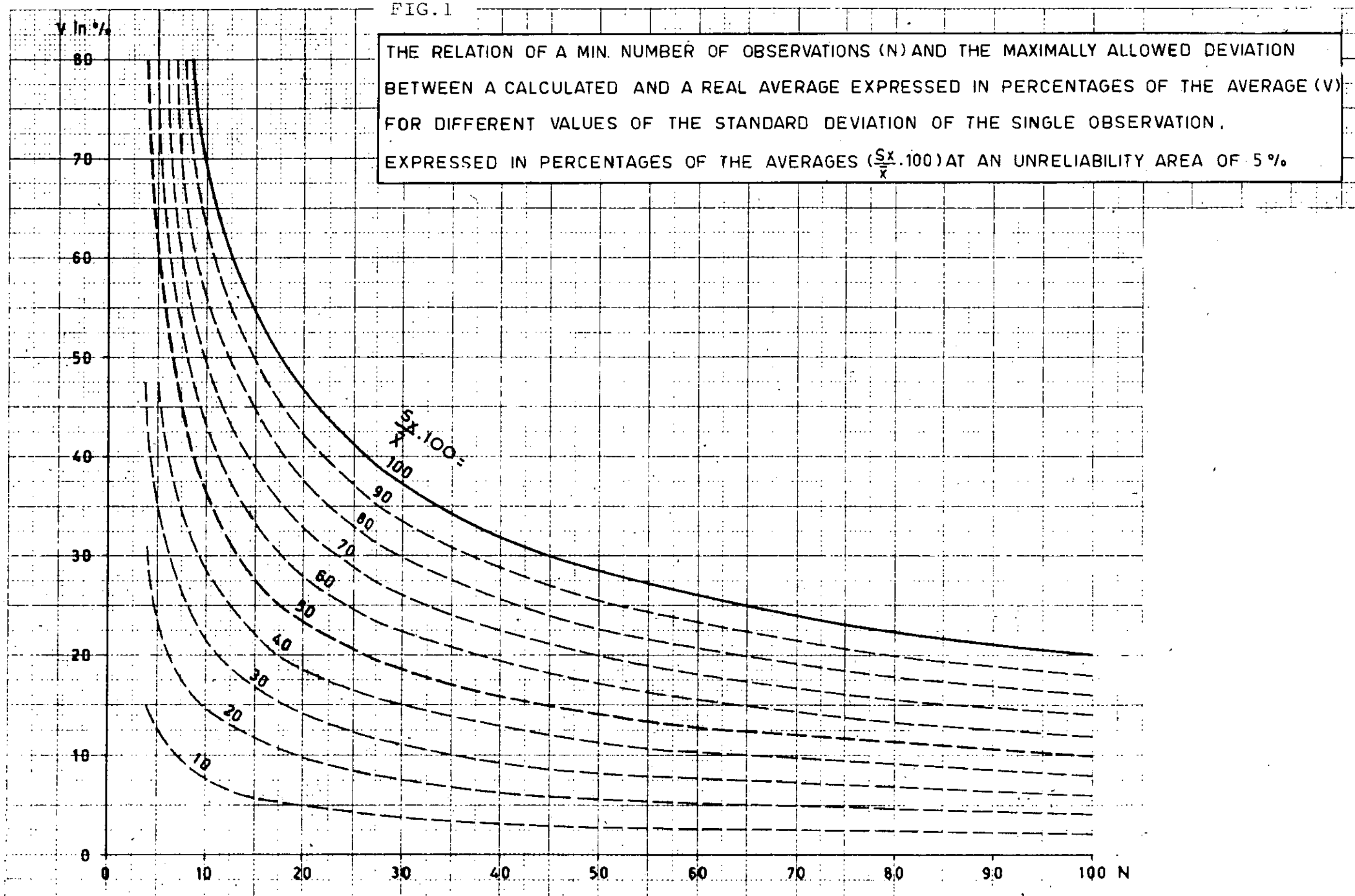
tn-1 = measure of excentricity dependent on the number of observations (n-1) and of α (measure of unreliability).

In this connection it is impossible to cut down the number of borings for the depth of the impermeable layer. If nothing is known of the value $\frac{Sx}{\bar{x}}$ 100 in Figures 1 and 2, it may be necessary to estimate this value by means of a number of permeability measurements made previously.

To determine the number of measurements necessary for budgetting, the drainage costs are assumed to be proportionate to the length of the drain pipes to be laid. Figures 1 and 2 can be used again here.

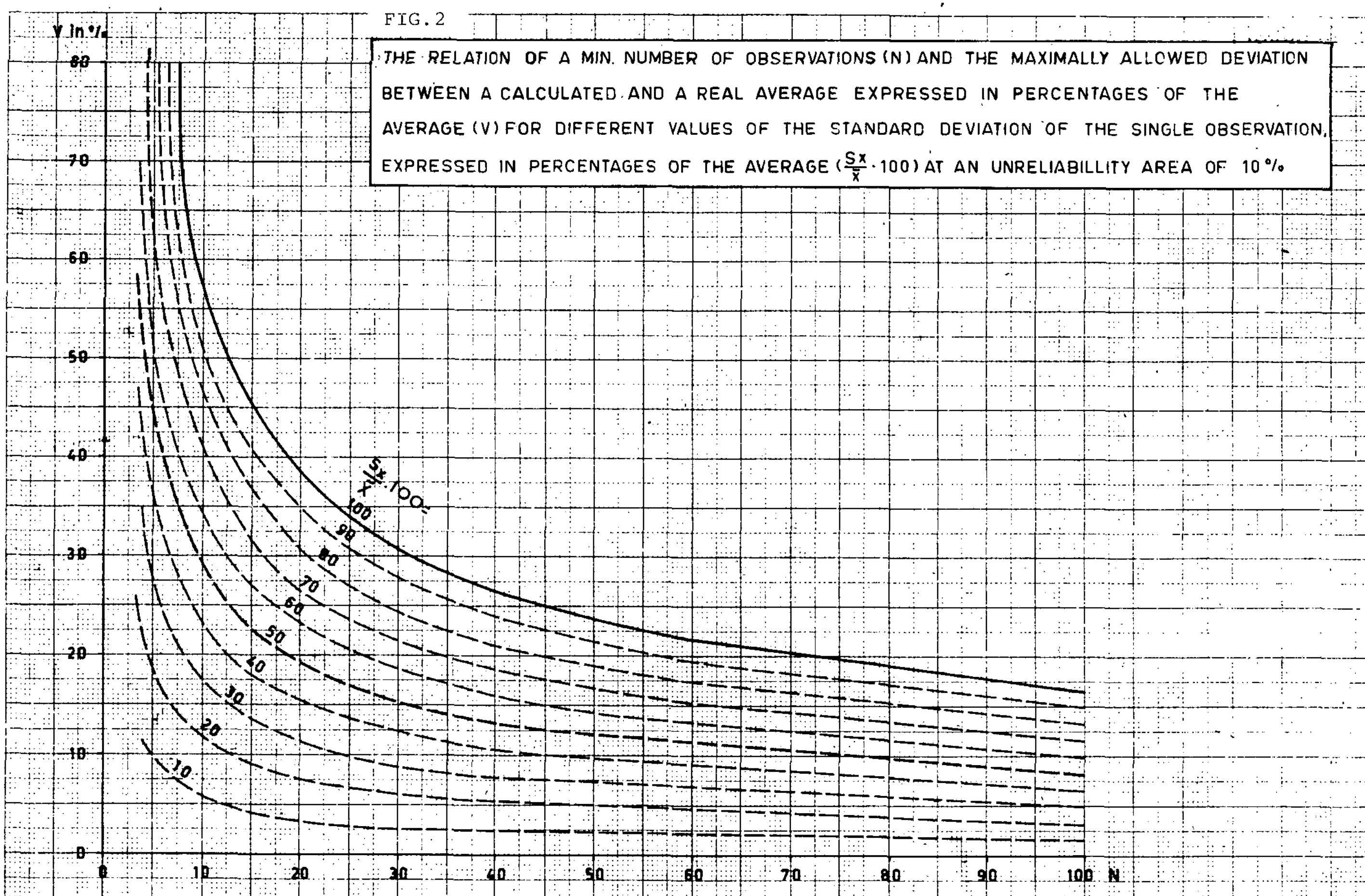
From the investigations at "Slochteren" it has appeared that, if a classification in profile types is used, the total number of permeability measurements (from Figures 1 and 2) can be reduced by about one half as compared with the situation in which no use is made of this classification. If the number of measurements were not distributed over the profile types in propor-

FIG.1



THE RELATION OF A MIN. NUMBER OF OBSERVATIONS (N) AND THE MAXIMALLY ALLOWED DEVIATION BETWEEN A CALCULATED AND A REAL AVERAGE EXPRESSED IN PERCENTAGES OF THE AVERAGE (V) FOR DIFFERENT VALUES OF THE STANDARD DEVIATION OF THE SINGLE OBSERVATION, EXPRESSED IN PERCENTAGES OF THE AVERAGES ($\frac{S_x}{\bar{x}}$.100) AT AN UNRELIABILITY AREA OF 5 %

FIG. 2



THE RELATION OF A MIN. NUMBER OF OBSERVATIONS (N) AND THE MAXIMALLY ALLOWED DEVIATION BETWEEN A CALCULATED AND A REAL AVERAGE EXPRESSED IN PERCENTAGES OF THE AVERAGE (V) FOR DIFFERENT VALUES OF THE STANDARD DEVIATION OF THE SINGLE OBSERVATION, EXPRESSED IN PERCENTAGES OF THE AVERAGE ($\frac{S_x}{\bar{x}} \cdot 100$) AT AN UNRELIABILLITY AREA OF 10 %

tion to the relevant area, but were based on an optimum distribution, the total number of observations would be reduced by about another quarter.

If the soil has been mapped out to a depth of at least 1.50 m with an estimation of the permeability factor, in most cases measurements will be hardly necessary to determine the budget. Nevertheless, to obtain a broad survey of the depth of the impermeable layer, in order to calculate an average drain distance, still deeper borings will be necessary.

3. The need of a pre-drainage investigation

Once it has been decided that sub-soil drainage will be included in the plan of provisions of a land consolidation area, the question arises to what extent pre-drainage investigation will be necessary. This depends on the information available about the condition of the soil (its degree of homogeneity), the deeper subsoil (deeper borings, geological maps) and the development plan itself.

Pre-drainage research by means of the Auger-hole method is less suitable for well-permeating sandy soils in which great drain distances are to be expected or when drainage is used instead of ditches by field enlargement. In areas with a very homogeneous soil (particularly with respect to the subsoil) a simple field research will generally succeed. If further investigation is required, its form and extent will be determined by the availability of soil maps. Various possibilities will be dealt with in the next section.

4. Soil type information and investigation method

4.1 If a soil map is available

The soil map of a land consolidation area, either rough (scale 1:50,000) or more detailed (scale 1:10/15/25,000) will be made in the preparation phase and provides a soil classification based on the profile structure to a depth of 0.80 m. Although the boring depth was generally 1.20 m, the soil map does not always supply information about the soil profile deeper than 0.80 m; only if a subsoil map or a sand depth map is made, we can obtain more information on this matter.

By an interpretation of the soil types due to hydraulic characteristics of the subsoil, the soil map can be more or less reduced to a (rough) classification in profile types.

When planning the costs of sub-soil drainage it is important to know the total length of drain pipes and, in a lesser degree, to know the differentiation in drain distances.

A reduction of the soil map to a number of profile types will often still supply a vague and incomplete picture, particularly as for the depth of the impermeable layer. It is, therefore, desirable to have borings made to maximum 2 or 4 m or, to put it more generally, till about 1/8 of the drain distance to be expected. The number of borings and its distribution over the area is dependent on the degree in which (hydrological) profile types have been distinguished and marked off. The scale of the map and personal insight will also play a part in this matter.

It must also be taken into account, that during the second phase (drainage plan), after re-allotment has been put to the vote, a more systematic investigation into the depth of the impermeable layer can take place.

When those borings are made, attention should also be paid to the profile characteristics. More research for the first phase (budget) can then be restricted to one or more trial blocks for every profile type. In every trial block a few drain distances will be calculated, after which the average drain distance can be extrapolated in proportion to the surface of each profile type. Dependent on the required reliability of the conclusion, the total number of observations can be approached statistically (Section 2).

To make a drainage plan which indicates and marks off the differences in drain distances as clearly as possible, the preceding phase has already provided a firm base.

Obscurities in the (rough) profile type classification, especially in the less homogeneous areas, have still been insufficiently resolved by the extra information from the additional borings and measurements. A closer investigation must, therefore, be focussed particularly on the depth of the impermeable layer, while permeability measurements should follow afterwards. The number of measurements (including those from the first phase) and their distribution over the profile types can, if required, be calculated statistically, and is highly dependent on the number of different profile types.

In a general sense it must be noted that deeper borings are significant especially in sandy soils. Clay soils are different because, on account of their relatively low permeability, a determination of the right depth of the impermeable layer has less influence on the drain distance in this type of soil.

If there is no detailed soil map or if an existing soil map does not give any information about the layers, deeper than 0.80 m, a quite different approach to pre-drainage research may be chosen in the first and second phase. In the first phase drain distance observations have been executed by means of systematic network of borings made with a average intensity of one boring in every 30 to 50 ha.

After the data have been worked out and soil data have been interpreted, additional investigation for the drainage plan can be directed at a more accurate marking out of different drain distance sections. Such an approach seems, therefore, to be only significant for areas with a more homogeneous soil. In heterogenous areas, the number of necessary observations will rapidly increase.

4.2 If a soil map is still to be made

The advantage of this situation is, that the soil survey program can be adapted to the future pre-drainage research. The extra costs involved are limited, in comparison with the total costs of the usual survey and certainly acceptable in view of an economization of the pre-drainage research to be made later on.

The items of survey extension are:

a) Boring depth and profile description should be extended to a depth of at least 1.50 m.

b) An estimation of the permeability of every different profile layer (beyond 0.50 m) from soil characteristics. Previous to the survey a rough inventarisation should be made of the most relevant profile layers, the permeability of which should be measured by means of the Auger-hole method.

The results of these gauge measurements should be analysed very accurately in order to gain some insight into permeability determining characteristics of the soil profile.

c) Although it is well-known that permeabilities measured in peaty soils may greatly differ, it has appeared from experience that the type of peat and its structure play an important role. Attention should be paid to this when surveying the soil. An estimation of the permeability will then lead to a better result than measuring.

d) A part of the borings can be made to 2 or maximum 4 m. The intensity and depth of the borings depend on drain distance to be expected. A careful consideration of extra costs of surveying on the one hand and a limitation of a later additional investigation on the other is desirable. Sometimes it is enough to make deep borings in a number of section lines.

From a profile type classification, a classification of (hydrological) profile types can be made by an interpretation of the permeabilities estimated, profile description and data about the depth of the impermeable layer.

Frequently there will not be enough information about the deeper subsoil, which may be investigated separately, if necessary. Together with these additional data sufficient material has been collected to make the budget for the subsoil drainage plan. A comparison between measured and estimated permeability factors is also necessary at this stage.

The step to the second phase, the drainage plan, can be restricted after the previous elaborations. For a proper differentiation in drain distance and a clear marking out of different distance sections, another number of measurements may be necessary and certainly a more or less elaborate network of deeper borings. No extra research is necessary in peaty soils and in profile types which represent a smaller area.

It should also be mentioned that a detailed differentiation in drain distance is not reasonable; a reduction to a restricted number of different distance sections, therefore, is preferred.

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Paper 1.06

DETERMINING HYDRAULIC CONDUCTIVITY WITH THE INVERSED AUGER HOLE AND INFILTROMETER METHODS

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Summary

The infiltrometer and inversed auger hole methods are briefly described and the results obtained with these methods are compared with those obtained with the auger hole method or calculated from the relation between hydraulic head and drain discharge. Measurements were made on four soil types ranging from sand to heavy clay and showing no micro-stratification. It appears that the theoretical restrictions of the infiltrometer and inversed auger hole methods are of minor importance in practice in view of the large variation due to heterogeneity of the soil.

1. Description of the methods

Infiltrometer method

When wanting to determine the hydraulic conductivity of the various layers of a soil profile, one often encounters a situation where the groundwater table is say 2 to 3 m below surface, i.e. too deep to use the auger hole and piezometer methods for the measurements. In such cases the infiltrometer method can provide a solution.

The infiltrometer can be used at successive depths in a soil pit to estimate the hydraulic conductivity of the various layers. According to the law of Darcy, the infiltration rate of water in unsaturated soil under a cylinder infiltrometer can be written as:

$$V = K_T \frac{\phi + z + h}{z}$$

where

V = infiltration rate (lt^{-1})

K_T = hydraulic conductivity of the transmission zone (lt^{-1})

ϕ = suction at the bottom of the transmission zone (1)

z = depth of the transmission zone below the infiltrometer (1)

h = height of the water in the infiltrometer (1)

The influence of ϕ and h relative to z diminishes as the depth of the transmission zone and the moisture content of the soil increases. The hydraulic gradient then tends towards 1 and the infiltration rate becomes constant, attaining the basic infiltration rate. Thus for wet soils we may write:

$$V \simeq K_T \simeq K_S$$

Although the basic infiltration rate is theoretically not equal to the saturated hydraulic conductivity, it nevertheless yields a fair approximation. The infiltrometer method can therefore be used to determine the order of magnitude of the hydraulic conductivity. One should keep in mind that the value obtained in a layered soil only applies to the layer penetrated by the infiltrometer, since lateral flow will occur below this layer if the hydraulic conductivity of the underlying layer is low. The main disadvantage of the infiltrometer method is the necessity of digging soil pits to install the infiltrometer.

Inversed Auger hole method

The inversed auger hole method, 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.

The surface over which water infiltrates into the soil at time t (Fig.1) equals:

$$A_t = 2 \pi r h_t + \pi r^2$$

Supposing that the hydraulic gradient is approximately 1, we may, according to the law of Darcy, write:

$$Q_t = K A_t = 2 K \pi r (h_t + r/2) = - \pi r^2 \frac{dh}{dt}$$

Integrating between the limits t = o, h_o and t, h_t and rearranging gives:

$$K = 1.15\, r \frac{\log(h_o + r/2) - \log(h_t + r/2)}{t} = 1.15\, r \tan \alpha$$

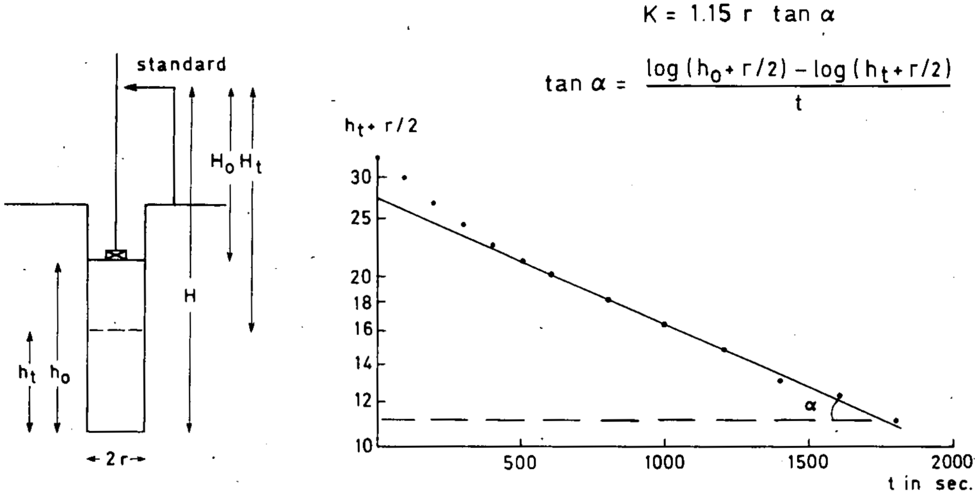

Fig.1. Inversed Auger Hole Method.

By plotting $(h_t + r/2)$ against t on semilogarithmic paper we obtain a straight line with a tangent α (Fig.1). The deviation of the first observations from the straight line may be due to unsaturated flow and a hydraulic gradient larger than 1.

In general the auger hole should be filled with water 1 to 3 times on loam and clay soils, depending upon the moisture content of the soil, in order to obtain a difference of less than 10 to 15 per cent between the successive measurements. On sandy soils it may be necessary to repeat the measurements 3 to 6 times.

The advantage of this method over the infiltrometer method lies in the difference between digging soil pits and making auger holes. Moreover, 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.

2. Comparison of results

The inversed auger hole and infiltrometer methods were used on several soil types ranging from sand to heavy clay. On three of the four soil types it was also possible to apply the auger hole method in periods of shallow groundwater table. On one soil type the relation between hydraulic head and drain discharge could be used to calculate the hydraulic conductivity of the soil profile. It should be noted that none of the four soils was characterized by micro-stratification.

The results obtained on three soil types are summarized below. As the hydraulic conductivity, in general, shows a skew frequency distribution, the geometric mean was used to represent the average of n measurements.

Soil type	Inversed Auger Hole			Infiltrometer			Auger Hole		
	n	K in m/day		n	K in m/day		n	K in m/day	
		Range	Mean		Range	Mean		Range	Mean
Sand	12	1.2-8.9	3.1	12	1.5-8.3	2.9	-	-	-
Loamy sand	20	0.3-3.0	1.1	10	0.5-4.0	1.3	20	0.3-6.5	0.9
Silty clay loam	16	0.2-2.5	0.7	16	0.2-3.0	0.9	16	0.3-2.2	0.8

For the layer from 40 to 100 cm in a heavy clay soil (80% <2μ), the relation between hydraulic head and drain discharge yielded a hydraulic conductivity of 0.3 m/day, ranging from 0.05 to 1 m/day on 14 plots. Below this depth the soil could be considered impermeable for horizontal flow, which means that the hydraulic conductivity below 100 cm was much lower than in the upper part of the soil profile.

The inversed auger hole method yielded a mean value of 0.3 m/day for the layer 25-75 cm, ranging from 0.1 to 0.6 for 16 observations. The hydraulic conductivity measured by the auger hole method equalled 0.07 m/day for the layer 100-200 cm.

The infiltrometer method was also used to compare plots with and without a gypsum treatment. The basic infiltration rate in the surface layer equalled 0.2 m/day for plots without gypsum and 0.35 m/day for those with gypsum.

It appears from these observations that on a wide range of soils showing no micro-stratification, the results obtained with the inversed auger hole and infiltrometer methods agree quite well. They also agree with those obtained by the auger hole method or calculated from the relation between hydraulic head and drain discharge.

Although the measurements pertain in principle to unsaturated soil and the assumption of a hydraulic gradient of 1 is an approximation, these theoretical restrictions are obviously of minor importance in practice, provided the measurements are continued until stable values are obtained. Besides, the heterogeneity of the soil has more influence on the average value than the theoretical restrictions.

In view of the large variation between the individual values due to the heterogeneity of the soil, it is stressed that the number of replicates should be sufficient to obtain a representative average.

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Paper 1.07

NOTES ON THE APPROACH TO DRAINAGE DESIGN

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Summary

The complexity of possible drainage solutions is smaller than the input parameters that make a large number with many of them unknown, and many of them of statistical nature. The size of projects often does not offer a design procedure by the "book". Without special development of methods for investigation and design, the engineering is left in effect to the contractors. The present theoretical knowledge is sufficient to produce the required methods. Some examples of design shortcuts are: hydraulic measurements in a full-scale drain; execution in stages; on the job design; the preparation of diagnosis and design guidebook based on regional experience; redefinition of the task of various regulatory drainage communities.

1. General

The following may not add up to a complete philosophy or well seasoned cook book. Nevertheless, it points out the need, at least in some places, for revisions in engineering training, professional aids and possibly in administrational frameworks. This is in view of what we know and do not know on underground drainage.

2. The complexity of the system

There are very many phenomena that may be related to the final act of drainage installation. They are in the physics of flow, in electrochemistry of soils, in microbiology, in climatology, in hydraulics, materials and, of course, in plant growth, and mechanical activity on the soil. These are besides various aspects of organisation and economy.

In conclusion: The number of parameters involved is extremely large as compared with most engineering fields of application.

In addition to the above complexity, the system with which a drainage engineer has to work is non-uniform at any scale level both in time and space. The non-uniformity involves:

- The climatic condition
- The properties of the soil
- The economical activities on the land
- The stock of existing installation and various human constraints

This non-uniformity means:

- That every field poses a somewhat new and different problem
- Great difficulties or even impossibility in obtaining a complete enough set of data for a rational design

The non-uniformity of the medium is probably the most fundamental property the drainage engineer has to cope with and it therefore calls for special methodology. This methodology should be the subject of a fundamental and lengthy discussion that is beyond the present scope.

For the following one concludes:

- Data on a micro-scale should be averaged out on the same scale or size as the necessary engineering elements
- The drainage engineer will often meet different conditions even on the average
- Rarely, if ever, will the drainage engineer have sufficient data "by the book"

3. The design degrees of freedom

The design degrees of freedom are by far more limited than the processes and constraints involved in a complete understanding of the system. The number of possible engineering solutions is limited by

- The target and basic diagnosis
- The number of principally different techniques
- Local, clear cut, constraints due to factors such as the soil, the economical activity topography, etc.

- The available executive facilities
- Administrational limitations.

Examples are in place. The first two types of constraints are quite clear. One can list only few drainage techniques such as, surface drainage and border channels, parallel underground drains, containment of clearly defined water sources, deep pumping by wells, vertical drain cutoffs. This constitutes more or less all principal possibilities. Certain basic diagnosis and targets rule out a priori various design possibilities or dictate drainage intensity, depth and reliability. Local conditions that are relatively simple to obtain, such as permeable and impermeable layers, groundwater table level or outlet, etc., artesian state or deep leakage, further limit the possibilities and direct towards the solution. The size of the project, its accessibility, the availability of contractors with various execution techniques influence the eventual result far more than the detailed understanding of many intricate processes:in many cases institutional support that determines certain credit levels or approve or disapprove of certain procedures, limit the choice of engineering alternatives almost to a clerical work. Thus, they may influence the drainage solution far more than a lengthy and expensive in situ investigation.

Conclusion: There is a contrast between the complexity of the system on one hand and the relatively limited number of available techniques or solution alternatives.

This contrast calls for a special preparation of the drainage engineers, the text books and the handbooks.

The effect of the above is different at different levels of treatment. There must be a distinction between

- ad hoc solutions of a limited or a small problem
- regional planning, prototype for a repeating problem or one of a wide scale
- development of new application methods and refinement of drainage techniques
- research and investigation which may be tied with a given local but may lead to more general conclusions.

Each of these levels differ by the following:

- Time available for investigation
- Extent of justified investment in investigation
- Opportunity to learn from experience
- Administrational freedom to choose various designs
- Opportunity to introduce unorthodox practices
- Availability of competent professionals.

It is unfortunate that most of the texts and teaching courses make little or no distinction between the different levels. They usually shoot too high and leave many practical problems of the lower level unsolved.

Only few drainage experts master at the same time the very complex theoretical background, the routine information necessary for a practising engineer, and some systematic approach for practical shortcuts or trouble shooting. Too often the practising engineer is puzzled at questions which have answers or should not have been asked at all. Somehow, the well-tested theoretical and experimental knowledge has not been translated into a simple minded set of rules.

Following are some such problems that often arise:

- How far downstream does an interceptor affect a drainage? This is an irrelevant question about a fictitious entity that has never been proved, long disproved and still a legend; single drain has an almost symmetrical effect (somewhat better upstream).
- Should the drains be parallel or normal to the contour lines? Few engineers will admit with certainty that it makes little or no difference as far as the underground soil water flow is concerned but it has some straight forward effects on the hydraulic design and ease of execution.
- Is there water feeding from the outside into the field? How important is it in terms of parallel drainage design?
- What are the criteria for salt leaching by drains? Is there such a thing as a control depth?
- In heavy soils is it useful to deepen the drains?
- Can one measure with common piezometers and permeameters the hydraulic characteristics of a very heavy soil? (Usually not.)
- A fast drawdown, is it a unique criterion that the drainage system is in place and works well? The lowering of a watertable is rarely a target. No drainable porosity is often associated with fast drawdown.

- There is flow out of the drains; evidently the system works, so why is there no lowering of watertable?
- A filter is expensive - how can one decide when to install it and when not?
- A mound in groundwater table - does it mean a local source of water or a non-steady flow?

Questions like these are mostly circumvented by the practising engineers, or are wrongly answered.

Conclusion: The existing theoretical knowledge should be reduced into simple rules concerning design parameters. This is necessary, especially for the practising engineers who have less time and means at their disposal to deal with any one field problem. This is in contrast to the fact that usually they have less preparation in formal learning and meet a larger variety of complicated cases. For their sake the fundamentals of drainage theory should be reduced to simple and clear elements, easy to understand and use.

4. The cost of drainage projects and design methods

The cost of underground drainage must be roughly limited to what the added production can pay or the saving on alternative expenses. Fortunately the prices of such drainage systems come close to this requirement. Consider now a farm of 10 hectares that requires drainage. The cost of such a drainage may be at most 100,000 Israeli pounds (usually less because of the fixed government support that did not follow inflation fast enough). At a reasonable rate of 5-10% engineer's fee, this means 5-10,000 Israeli pounds which constitutes not more that 5-10 working days with expenses.

This should cover:

- Preliminary investigations;
- Surveying;
- Preparation of plan and detailed design;
- Tender and contract with contractors;
- Inspection of execution;
- "As made" surveying and plotting;

- Appearing before a committee for the approval of the plan and the financial aid (including lobbying at early stages of the planning).

Naturally it is impossible to do all this properly for the allocated design fees. There is a natural trend then towards one or several of the following solutions:

- Routine design that has very little to do with the special local problem;
- The creation of unwritten or written standards that have favour with some clerk at a crucial point of the system;
- Design by a contractor (although covered up by some engineering rituals).

This is what happens unless one faces the question and produces design tools methods of trial and error, and flexible enough administration rules.

At the present state of the art the drainage practices are determined mostly by contractors and by government officials while the investigation and design stages are more of a secondary importance and are often maintained more like a ritual.

In some drainage projects although of a much larger scale than the above example, the preliminary investigation reaches 50% of the cost! In a dynamic society where officials change, and regional plans change, projects have been known to be re-designed several times. In the case of drainage it is often better, faster, and cheaper, to increase the factor of safety by 2 than fiddle with endless investigation and re-designing.

The committees that are supposed to approve or disapprove projects would do far better to consider their task as that of accumulating experience, digesting it and disseminating it for the general benefit.

In view of the above there seems to be a need for new investigation and design shortcuts to cope with the complicated drainage problems within limited time and means.

5. Some examples for shortcuts to efficient investigation and design

The examples in the following are far from exhausting the subject. Probably the number of tricks is as big as the number of good experts that convene here (in the Drainage Workshop).

Guide for the drainage engineer

Beyond the existing textbook material such guides are necessary. They may be better prepared, or at least revised on a local basis. Such a guide should start with a simple diagnosis of the problem, then simple methods of diagnosis validation, and the collection of easily obtained information. Alternatives of design should be introduced very early in the guided path. One should introduce "shorthand" or modular design that include standard elements and a system, as it is the practice in mechanical engineering and electrical engineering. Finally, a list of simple investigation methods should be suggested that are directed for specific design decisions. Any method of investigation which does not lead to such a decision or is not the simplest to lead to such a decision should be overlooked.

Sources of information

The best source of information is a neighbouring project or previous experience. Often an existing channel allows the observations of drawdown, improved yields, leaching distances. However, great care must be exercised in the accumulation of experience by the most critical experts and methods. Too often regular prejudice or habits are mistaken for a well-tested experience. Too little effort is given to learn from experience in comparison with means specially devoted for research and local investigation.

Methods of hydraulic measurements

The best, and often the fastest and cheapest method to measure the hydraulic conductivity is by digging one or two trenches and measuring the drawdown at several distances.

A series of point measurements of the hydraulic conductivity, the porosity, or drainable porosity, is a very favourable procedure by many textbooks and manuals. Such point measurements are eventually to be combined into some drainage formula. The error can be half an order of magnitude despite a very tedious procedure of sampling, the use of an auger hole method, etc.

Any reasonable drainage formula (and there is quite a number of them in steady and non-steady cases) is the result of several steps.

- Flow equation such as Darcy's law
- Law of conservation that leads to a differential equation
- Boundary and initial conditions including assumptions about the medium's properties
- A solution, exact or approximate, which is the drainage formula.

Any such solution can serve also as a method for measuring the hydraulic parameters of the medium. The assumptions made to reach the solution may be quite coarse. The design decisions using this solution will be excellent as long as the parameters have been found by measuring with the same solution.

Actual examples are beyond the scope of this article. It would, however, be a relatively easy exercise to device ones own. Usually, a non-steady flow formula will require the measurement of drawdowns in a piezometer over several times and preferably in several distances from the drain. Parameters, such as the transmissivity and drainable porosity may be found by best fit. An extreme change in conductivity with depth and extreme unisotropies may be better detected by using formulas that assume them. Sometimes drains of more than one depth may be used as a precaution.

The overall measurement of parameters led on one field to half an order of magnitude (about 5-fold) higher transmissivity as compared with the results of an auger hole which were themselves several times higher than the so-called undisturbed samples.

The overall measurement can be aided by flooding or by an irrigation system to elevate the watertable. Surely this cannot work in a remote area where over irrigation is anticipated to raise the watertable in several years. But then it would be extremely questionable to assume the hydraulic behaviour of a drainage system on the basis of some hydraulic tests on samples if they can run into 5-fold error and more.

Samples are used more effectively to identify the type of soil information on hand and look for a similar one where experience has already been gained.

Experimental execution in stages

The above discussion leads us to a very important design approach by trial and error. First stage execution of a drainage scheme can serve as an experimental for a second stage. The embodiment of this principle can be obtained in several ways. Here is one. Consider the following table of initial spacing and the number of drains inserted in between at later stages.

First stage spacing in meters	100	80	60
Number of in between drains of the second stage (distance in meters)	1 (50)	1 (40)	1 (30)
	2 (33)	2 (27)	2 (20)
	3 (25)	3 (20)	3 (15)
	4 (20)	4 (16)	4 (12)
	5 (16)		

Clearly one can gain a high flexibility in the design of the second stage on the basis of the first stage. Assume for example that the distance between drains is found to be 20-30 meters. If the initial distance was 100 meters the maximum error in final distance would be halfway between 25 and 33 (3 and 2 interdrains). This is 4 meters error or about 16%. This is an error far smaller than any error found in the measurement of soil parameters and estimating climatic conditions.

On the job design

A good drainage engineer should be able to leave some decisions for the time of execution. A framework design should exist beforehand and alternatives for decisions should also be lined beforehand. There is nothing like opening a trencher line for a thorough soil survey at length and depth. In a matter of 24 hours drawdown result can also be computed and translated to a decision. The major framework of elevations could be fixed beforehand. There is not much further freedom in the laying of tiles between these elevations.

Such design methods should be worked out. Engineers should be trained for them. However, most important, the administrational frameworks must also be changed in the process of checking and approvals and financial support and on-the-job inspection.

6. Conclusions

Much more good can be obtained by adaptation of existing knowledge to proper practice than by a new research.

A different type of training of the drainage engineers should be based on a good understanding of the phenomena involved on one hand and on existing practices on the other. However, the emphasis should be put on a systematic approach for a fast and efficient elimination of design alternatives. Commitees and commissions for drainage project approval and all other functioners cost today more than all the possible savings which are to be obtained by their close observance. Rather, their main function should become that of accumulation and dissemination of experience and know-how. If the above changes will not be made with the help of proper guidebooks, the design of drainage projects will be done in effect by contractors and will be based often on habits, superstitions, and prejudices, rather than on really measured and checked experiences.

Paper 1.08

CHOICE OF A FIELD DRAINAGE TREATMENT

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Summary

This paper is a general review of water excess situations occurring in France and briefly describes the different field drainage treatments recommended by the Ministry of Agriculture.

Special attention is paid to the determination of the origin of excess water and the need for pedologic and hydraulic soil data.

1. Introduction

Because of the different conditions of soil, climate and crops, there is no unique solution for field drainage. It is first necessary to separate the case of local drainage (source and seepage areas) from the case of drainage of a whole field.

In the second case, theoretical considerations have shown, for the last fifteen years, the importance of determining the physical and hydraulic soil characteristics (permeable layers thickness, hydraulic conductivity, specific yield). Hydrological and economical data are also needed. For example, economical considerations can lead to choosing new techniques, such as moling or subsoiling, when drainage by tile drains alone would be too expensive.

2. Different aspects of excess of water in fields

The main origin of water is the rain falling on the catchment area. The average rainfall of a catchment area can be divided into three parts:

- the run off which runs on the surface of the soil down the slope to ditches and rivers;

- the infiltration water which percolates through the layers of the soil and contributes to its water supply or feeds the lower aquifers:

- the evaporation which represents the losses of water due to both the climate and the plant growth.

The schematic water balance is:

$$\text{Rainfall} = \text{Runoff} + \text{Infiltration} + \text{Evaporation}$$

$$\downarrow \quad \downarrow$$

$$\text{Drainage}$$

3. Evidence of excess of water

An excess of water appears in a part of a catchment as soon as runoff and infiltration do not play their normal part, i.e. when rain water cannot reach the natural terminal outlet soon enough.

Drainage problems arise from either of the following reasons:

pounding on the soil surface is commonly found on pans in the top layer of loamy soils; the more compacted the pan, the less the soil can take up water. The compaction is mainly determined by the machinery working conditions. Working in too wet a top layer increases the compaction, shuts off the layers by their loam particles, and allows the development of mushrooms on the surface that lengthens the normal evolution of the organic matter and soon reduces root growth. A solution can be proposed to the farmer to improve the infiltration rate by modifying his farming methods.

local saturation of the top layer, without a characteristic water table is observed in many clay soils. Rain water fills in the cultivated layer, though low layers of generally high density have low infiltration capacity. That type of water-logging arises essentially on undisturbed old geologic formations where the slope is the main reason for subsurface water movement. In France, it is spread on old primary mountains: Lorraine, Massif Central, etc.

seepage, i.e. spotted seepage, spring seepage lines - have an extremely varied geologic origin often difficult to clearly make out.

waterlogging with a water table developed on an impervious layer can be easily checked by digging an auger hole. The water fills it from its bottom up to a static stable level within a time varying from 10 minutes for very permeable soils (sandy soils) to several hours for impervious pedologic formations (loamy or clay soils). The level reached in the auger hole is the water table level. That one varies throughout the year, low or very low in summer during a drought period, it can reach the surface with excessive winter rainfall.

After a dry period, rainfall water filtrates down through the soil and increases its water content. The water movements are controlled by two forces: gravity and suction. Suction allows movements from high to low moisture content areas. The greater the water content, the less the suction is. Gravity pulls the wetting front down. When moisture content increases up to saturation, gravity movement becomes dominant.

At the experimental site of Arrou (Allee & Devilliers, 1975), drain flow appears after an average rainfall of 150 mm. An impervious layer can stop the drawdown of the wetting front.

But what is really an impervious layer? A totally impervious layer never exists in natural conditions. A layer is looked upon as impervious whens its permeability does not allow the filtration of water coming from the upper layer within a fixed time. We consider that layer B is impervious with respect to layer A when

$$K_{(B)} < \frac{K_{(A)}}{100}$$

In that condition a water table grows upon the the impervious layer and a saturation front keeps on getting up till the rainfall stops.

A flow arising from the slope of the fields can create springs or seepage lines. The water table can draw down slowly thanks to a small filtration through the "impervious layer". In most of the cases a water table disappearing at the end of spring or at the beginning of summer when evaporation is at its maximum is called temporary water table. At the opposite a permanent water table remains present in the soil all through the year (alluvial water table).

Flooding is always a temporary event caused by a river flood or by accumulation down a slope of an important runoff.

4. Origin of the excess of water

Two main situations are generaly given consideration (Feodoroff & Guyon, 1972).

a) *Water coming from the outside of a field* has either one or both of the two following origins:

Runoff: the greater the slope or the saturation of the soil, the more important the runoff.

Deep circulation of a confined or unconfined aquifer emerging in a seepage area.

b) *Water coming from the field to drain* arises from a small runoff and a bad vertical infiltration that both make a field drainage problem.

Classically four categories of seepage areas can be isolated:

Case 1: see Fig.1.

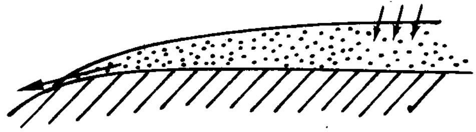


Fig.1. Emergence of an impermeable layer in a slope.

Case 2: see Fig.2

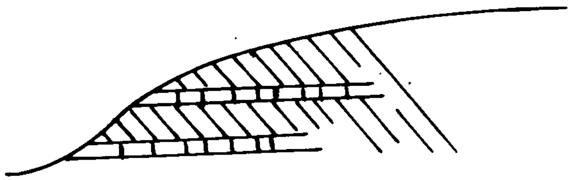


Fig.2. Emergence of a very permeable layer equivalent to a drain.

Case 3: see Fig.3

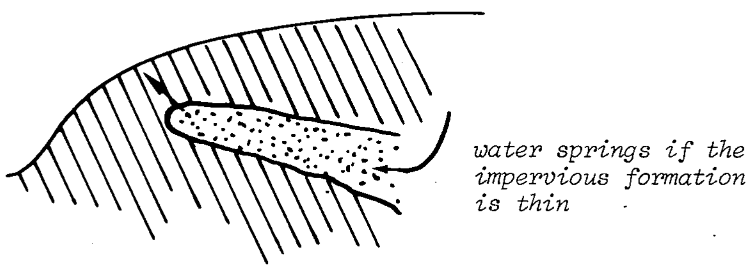


Fig.3. Confined aquifer with high piezometry.

Case 4: see Fig.4

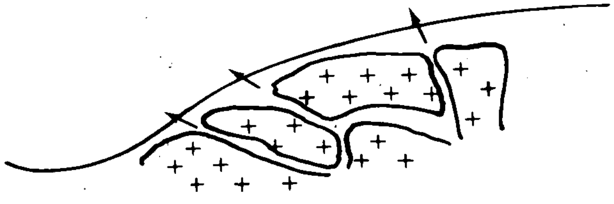


Fig.4. Confined or semi-confined aquifer in a fractured rocky formation. Springs are spread in a vast area and can move.

These specific drainage problems need specific drainage systems.

Open ditches or draining trench are dug to intercept seepage lines (see Fig.5).

Fig.5. Interception drains.

But the origin of water can be more complex and it can be rarely assume with sufficient accuracy before drainage begins. Random drains are scarcely effective and systematic super-intensive drainage leads to a very expensive scheme that never pays back. A french national company (Comp.d'Aménagement des Coteaux de Gascogne) successfully sets the rules for a good drainage of seepage:

-Open a trench through the seepage in its main axis and with the maximum grading deep enough to reach the aquifer. (Though it is always profitable it is not always possible to reach this impervious layer.) The trench is then lengthened, and lateral trenches are added to intercept the preferential ways of water.

Trenches are 45-60 cm wide and up to 3 meters deep, fitted with a good diameter pipe (ϕ 65 mm) and backfilled with coarse angular gravel (see Fig.6

Common rules of drainage work have to be applied in seepage drainage:

a good and regular grade of the pipe;
pipe diameter correlated to the flow (always larger than 1-2 l/s);
good ditch and outlet maintenance.

The average price of a seepage drainage is 700 FF but it can reach 2000 FF.

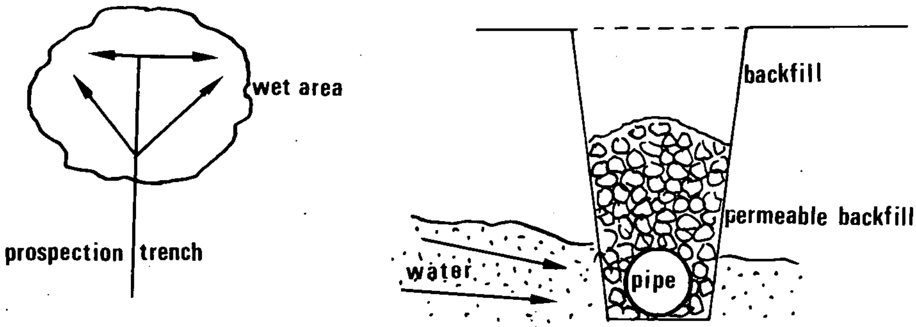


Fig.6. Trenches with backfill.

5. Drainage requirements

A good morphologic examination of a waterlogged soil gives lots of valuable information. The iron and manganese dynamics in the profile are positively correlated to water dynamics. The soil scientist can play an important role: he looks for the depth and the intensity of iron and manganic concretions. He gives the average depth of the impervious layer (when it exists). But these qualitative data must be completed with hydraulic measurements of the water table movements.

It is of great use to set piezometers in fields to be drained (Fig.7). A piezometer consists in:

- a perforated pipe (1-3 cm wide);
- permeable gravel along the holes of the pipe in order to decrease the response time;
- impervious protection above making the piezometer independent of the surface runoff or rain. A high overburdening pulverous clay (bentonite), sprayed very dry, shuts off the higher part of the auger hole rapidly.

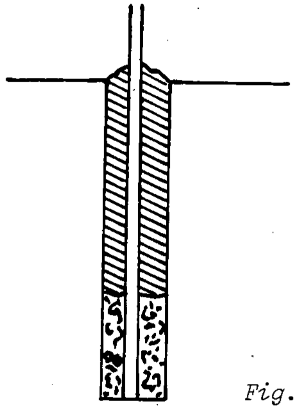


Fig.7. The piezometer.

It must be noted that these piezometers allow as well:

- hydrodynamic measurements (Guyon, 1971, 1976)
- detection of the impervious layer;
- continuous recording of the water table level.

As an example of a pedological survey of drainage requirements the chart established for the Pays d'Ouche (Devillers et al., 1975) is of great interest:

h_o : no pseudo-gley in any place in the profile

h_1 : pseudo-gley below 80 cm

h_2 : pseudo-gley within 50-80 cm

h_3^- : pseudo-gley beginning at 40 cm

h_4 : pseudo-gley at 20-30 cm

h_5^+ : pseudo-gley in the whole profile.

The piezometeric survey achieved during two very wet winters and concerning sites belonging to various pedologic situations shows a close relation between that chart and the waterlogging degree during high rainfall periods (see Fig. 8).

Water table evolution during 1974–75 winter in different sites corresponding to different logging conditions

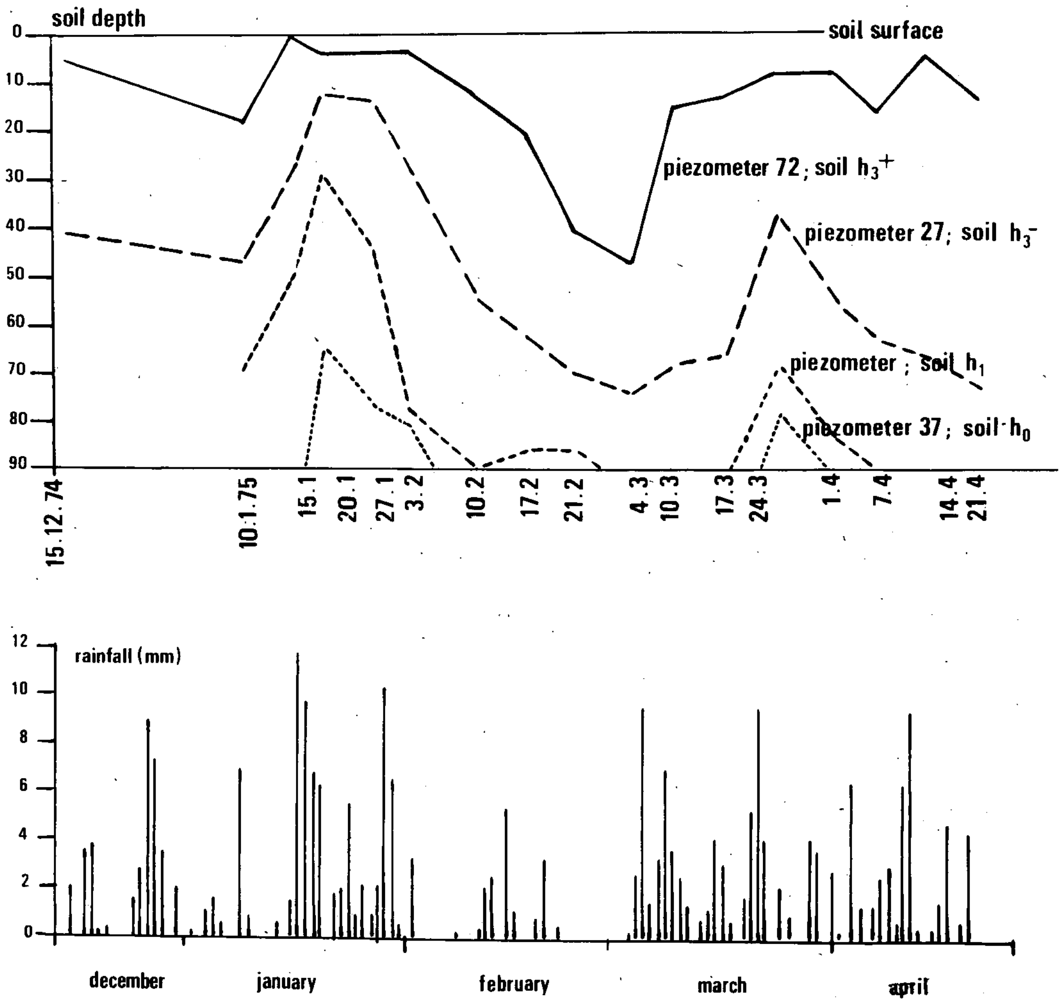


Fig.8. Water table hydrographs.

To all these "objective criteria", subjective criteria are added to give more detailed information on the situation. Farmers living with excess of water must do with specific constraints which can be divided into three categories:

a) Field preparation and soil working conditions

A saturated soil can reach the plastic - even the liquid - stage, characterized by small adherence and poor stability of field machines. The working periods become shorter. Good drainage must bring a high speed of the drawdown movement of water table - 25 cm a day beneath the ploughing depth after the rain stops.

In grassland cattle spoils when moving on a saturated soil (a cow's leg pressure reaches 2 kg/cm^2 standing and more than 6 kg/cm^2 moving). In loamy soils grass is protected when the water table is deeper than 50 cm.

b) Crops and crop system choice

Saturated soil conditions make it difficult to seed or harvest crops, resulting in low crop yields. The growth period (for barley corn, grass, maize) is shortened. Plants are subject to diseases that make them less resistant.

The yield losses can be modelled by parametric equations where a variable is the length of waterlogging period. Formulas exist which integrate both yield losses and water table drawdown speed (Guyon, 1970).

6. Drainage systems

The oldest system to drain is by various sizes of rigs and furrows. The most used french terms are "billons", "planche", "ados", etc. This system is easy to install at a low cost in a field and is very effective when drainage problems arise from flooding or ponding, but:

- it becomes difficult or impossible to mechanize field works (ploughing and harvesting);
- drainage is poor and irregular and does not provide a fast water table drawdown.

Such a method is valuable in extensive grassland or when more effective drainage is too expensive (i.e. in blocky-stony thin soils where the price of under drainage is more than 8000 F/ha).

Gutters

They consist of small ditches to intercept excessive winter rainfall. But the rootzone stays saturated during crop development. This system has been experimented for several years in "Charentes Maritimes" in dry flat soils and used to be the traditional drainage of the Flandres Maritimes in the North of France.

But:

- interception of excessive rainfall is just local, drainage is poor;
- ditches take surface off the cultivated area;
- it is difficult to mechanize agriculture.

Field underdrainage

It is first necessary to make out the difference between underdrainage with pipes alone and moling or subsoiling added to a pipe drainage scheme.

a) Underdrainage with pipes alone

Water movement in an underdrained soil depends on two families of parameters:

- *Soil parameters:* drainable porosity, hydraulic conductivity, depth of the impervious layer.
- *Topographic parameters:* natural terrain gradient, vegetation cover, sensibility of top layers to compaction or to pan formation.

The drainage scheme design depends on two other types of information:

- *Climatologic data*: average rain intensity, frequency and duration.
- *Economical parameters:* cost benefit elements (i.e. yield losses) to optimize the scheme.

b) Moling and subsoiling

In very compacted soils or in soils characterized by poor hydraulic conductivity, an extra drainage system is added, crossing the pipes of the underdrainage scheme, in order to increase, more or less quickly, both the effective depth and the permeability of the drained area. A classical drain spacing formula gives a good picture of the points played by these two parameters:

$$E = 2\ H \sqrt{\frac{K}{I}}$$

The bigger H or K the bigger the spacing.

So it is necessary to distinguish the following cases:

☆ *Deep and permeable soils:*

$H > 0.60$ to 0.80 m
and/or
$K > 0.25$ m/day (in situ measurement).

Good drainage is provided by regular parallel drains layed at a calculated spacing.

☆ *Thin or rather impervious soils:*

$H < 0.6 - 0.8$ m
$K < 0.25$ m/day.

Effective drainage is given by regular drain layout with secondary treatment:

★ *Moling* (in soils of regular texture with more than 30% of clay, a good structural stability and a gradient better than 2 $^{o}/_{oo}$ and smaller than 5%).

★ *Subsoiling* (in compacted soils, blocky soils, unstable soils, soils or irregular granulometric composition).

Permeable backfill is necessary with a secondary treatment. But experiments began a few months ago to try to make out the situations where it is not of real use.

7. Choice of a drainage system

a) The choice depends on:

☆ *the aim*, which is linked with the evolution of the crop system in the future (transformation towards more sophisticated systems, such as greenhouse production, intensive cereals).

☆ *technical criteria* made out by the soil scientist:

- origin of the excess of water:

  geological survey ⇨ interception / drainage

- hydrodynamic properties of the soil:

  underdrainage with no secondary treatment
  underdrainage with moling or subsoiling.

- topographic survey:

  gradient (general and local)

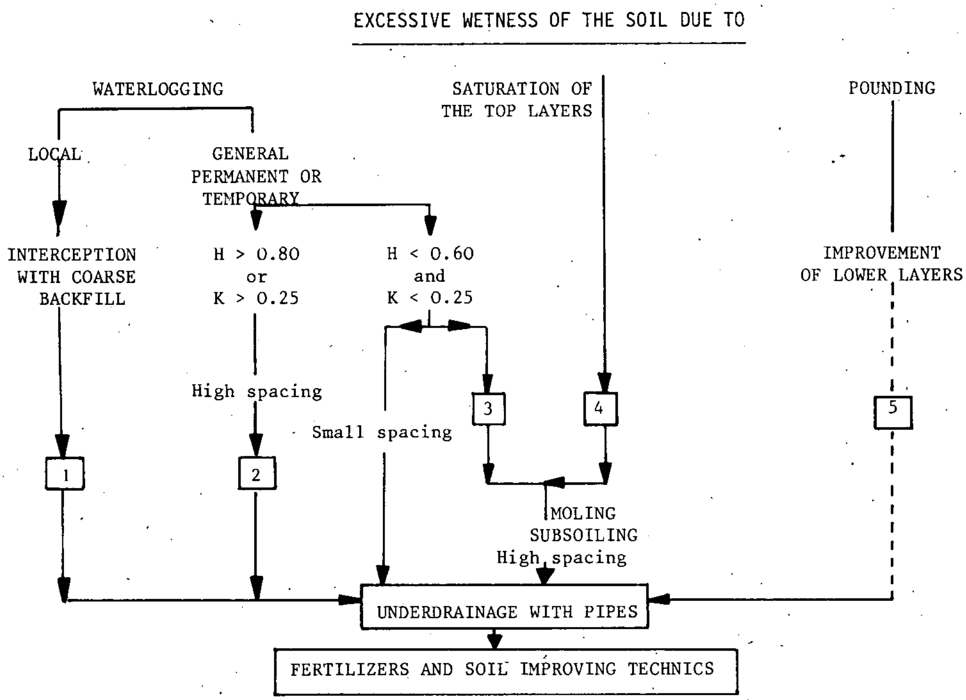
- efficiency criteria:

  clogging risk (sand, clay, iron ochre problems)
  maintenance of ditches, laterals, outlet.

☆ *economical criteria:*

- cost of the layout
- average estimated benefit
- grant aid, subsidies, ... either collective or individual.

b) Chart (Fig.9) summing up the choice of a drainage system (proposed by J.L Devillers, from R. Eggelsmann's suggestion, ICID European Congress, Sevilla, 1973).



☆ Situations:

1 Seepage spring-lines	2 Blocky sandy deep loamy alluvial soils	3 Thin loamy sandy alluvial colluvial soils	4 Clay undisturbed, thin loamy soils	5 Soils without typical hydromorphy but compacted

☆ Prescription:

Permeable backfill	Filters in sandy soils with less than 15 % of clay	Secondary treatment + permeable backfill

☆ Clogging risk:

Iron ochre problem either in organic and mineral soils or if water has a high iron content

Fig.9: Chart for selection of a drainage system.

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Paper 1.09

SOIL FUNCTIONS AND DRAINAGE

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Summary

First some general facts about climate, soils and drainage in Sweden are given. Especially the soil profile formation, the structural pattern and drainage properties are discussed. The Swedish drain test field program in the period 1947-1977 is shortly presented. Primary and secondary drainage effects have been followed on about 125 drain test fields. Some examples are given. The experience gained in this field test program gives the base for the choice of drain intensity e.g. drain space and depth in separate agricultural regions. In a second subsection the root growth and its dependence of the root environment are discussed. The third subsection on changes of functions of the soils as a result of compaction shows, using experimental data, how a number of properties of importance for the root environment is influenced, namely pore space distribution, hydraulic conductivity, air permeability, penetration resistance and root growth. The vulnerability of a soil to compaction is primarily related to the water tension acting on the soil skeleton. The primary way of regulating the moisture content and thereby to a certain extent the trafficability, is by drainage, as could be shown by the results of drainage experiments. One has, however, to realize that drainage can influence only a small part of the wide variation in soil water tension. The drainable pore space will be gradually destructed when raising the compaction forces in the soil to 200 kPa. This value will be stated as an allowable limit over which severe deterioration in the root environment and drainage properties will occur.

This paper is mainly a compilation of the authors papers in the Journal of Agricultural Land Improvement and the SIAE-Bulletin 354 written in Swedish.

1. Some general facts about climate, soils and drainage in Sweden

The climate in Sweden can be characterized as semi-humid. In the main agricultural areas normal annual precipitation is between 500 and 700 mm. Precipitation is normally much less than potential evapotranspiration in the spring and early summer, the reverse being true in late summer, autumn and winter (Fig.1).

The total arable area is 3.0 million ha. The main part of the arable soils is fine textured (clay, clay loams, silty and sandy loams). The remaining areas are coarse-textured and organic soils. Two third of the arable soils has a need of drainage and the main part is also drained with varying intensity. The intensity is based on the ratio between the costs of installing a drainage system and the benefits of better workability and trafficability and of less frequent and less severe yield depressions.

The intensity problems have been studied in a large field test program in the period 1947-1977 with about 100 drain test fields to study the drain spacing and about 25 fields to study the influence of drain depth. On these test fields in average 14 crop seasons have been followed. It means totally about 1800 crop seasons.

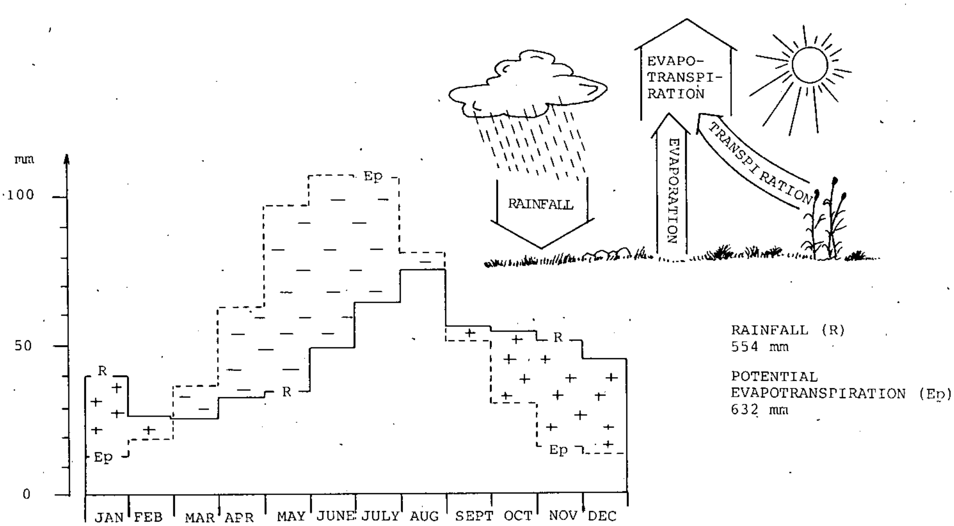

Fig.1. The relation between rainfall and potential evapotranspiration in eastern Sweden (Uppsala). The growing season starts in April (>5°C) and ends in October (<5°C) with a length of about 200 days.

Soil profile formation and average water holding capacities

The value of soils for crop production is determined by how well it can supply plant roots with water, air, and nutrients. The soil consists of solid, liquid, and gaseous materials, forms the soil skeleton and provides the structural pattern of the pore space. In an average cultivated soil, the pore space is about half of the total soil volume. Depending on the shape of the skeleton, the pore space is divided into a complicated system of channels and cavities; the pore system below a water table is completely filled with water. If the water table drops, the pores are progressively emptied of water and air fills the pores. A favourable condition for plant roots is created in the border zone between water and air in the soil space. The distribution of air and water in the soil profile is determined by the distribution of various sized pores, the location of the water table, and the addition and removal of water.

The particular pore system that forms is mainly dependent on the particle-size distribution of the soil material, i.e. texture or soil type. The clay fraction is especially important, and the organic matter is of great importance in the topsoil. The relationship between clay content and water holding capacity in Swedish cultivated sedimentary soils is pictured in Fig.2 (Andersson & Wiklert, 1972). The pore volume is at a minimum of about 41% in a light clay. It increases with increasing clay content to about 45% in very heavy clay. Clayless and loamy soils have also developed larger pore spaces than light clay soils.

In Fig.2, the pore space is divided into two main parts by the lines separating the unavailable from the available water, denoting the so-called wilting point. The amount of unavailable water increases steadily from about 2% in clayless soils to about 25% in heavy clay and 33% in very heavy clay. Thus, the available water decreases from about 40% in the clayless and loamy soils, to about 20% in heavy clay, and then remains quite constant between 18 - 20%. Some of this water is held in such coarse pores that they empty at a water table depth of one meter. The drainable water makes up most of the available water in coarse, clayless soils, but is limited to smaller amounts from light clay through the fine-textured clays. In subsoils, which the figure prefers to, the amount of drainable water lies between 2 and 7%. In topsoils with moderate organic matter contents, it lies between 5 and 10%.

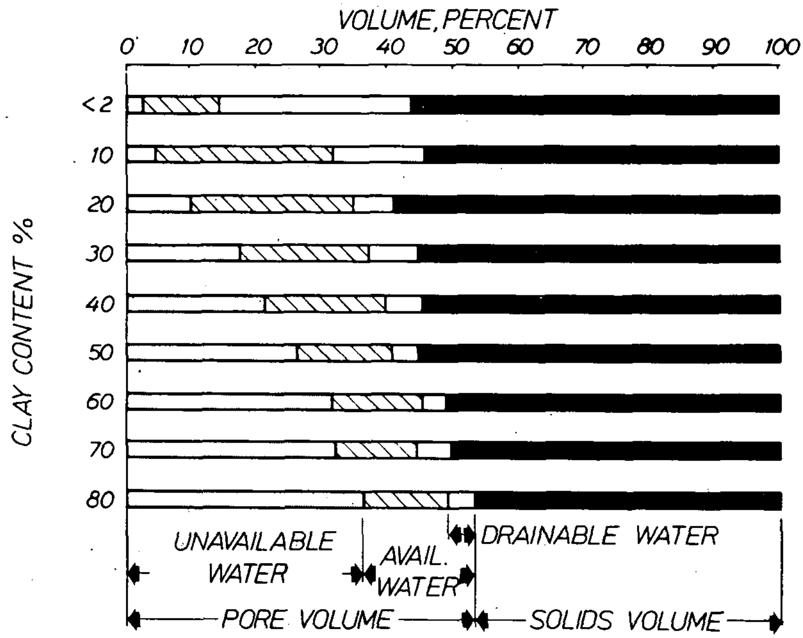

Fig.2. The relationship between clay content and water-holding capacities in Swedish mineral soils. The horizontal bars indicate solid and pore volumes. The pore space is occupied by water of different availability. The values are typical at the given clay content (according to Andersson and Wiklert, 1972).

The structural pattern and drainage properties

The concept of soil structure includes everything that deals with the fabric of soils, i.e. the manner in which individual particles are arranged and joined in large and small units to form aggregates.

The cementing agent is primarily clay particles and soil organic matter. As these particles are mutually cemented, they also bind together the larger silt and coarse-textured particles.

The structural pattern results from various structure generating factors which are illustrated in Fig.3. A summation of processes that will change the structure of a clay soil and, because of the properties of clay particles, will cause aggregate formation and maintenance of the structural changes, is also indicated in the figure. Small channels branch out from the larger cavities and channels (cracks, worm holes, and root channels). One differentiates between macrostructure containing macropores and microstructure containing micropores. An aggregated clay soil provides for rapid movement of water and air in the macropores. Likewise, the root system can spread out in the subsoil's network of cracks and channels. Therefore, the root system can reach, and effectively penetrates, large volumes of the subsoil and thus assure water availability during prolonged dry periods.

Coarse-textured soils that lack cementing substances and ability to form a macrostructure, with cracks and channels, are called *single-grain soils*. In these types of soils, root penetration is halted immediately below the topsoil because of such various reasons as mechanical resistance, a dry zone, lack of nutrients, or low aeration. Therefore, in this case the subsoil is not available for root growth; however, a portion of any stored subsoil water may be transported by capillary action to the rootzone. It is therefore necessary in single-grain soils that the topsoil structure provides for good root development and, consequently, for good root contact with the subsoil.

Structural boundaries in the soil profile

The presence of structural boundaries can be observed with the naked eye in prepared profiles (Fig.3). They can also be determined by measuring physical properties.

The boundary for cultivation effects

Several things are involved in influencing cultivation, one of which is compaction as a consequence of pressures from passage of vehicles and machinery. This limit is most clearly demonstrated in east Swedish clay soil profiles where a very definite structural change often occurs at 30- to 40-cm depth.

The frost boundary

Coincides with the normal frost depth. Frost action produces a greater granulation than simple drying. Near the average limit at a depth of 60 to 80 cm, there is a gradual transition to coarser aggregates, separated by cracks, that characterize the profile down to the drying boundary.

The drying boundary

The drying boundary is at that depth to which soil will normally dry because of water uptake by roots.

The structural pattern generates typical variation with the depth in the profiles, e.g. of the physical properties and thereby the root environment and the drainage properties.

In Table 1 some soil data together with results from 11 drain test fields are given. The fields are from the glacial plains in middle Sweden. The clay content varies between 40 and 80%, except in field 56, a sandy loam. The drainable pore space varies between 2.2-7.5% for the clays.

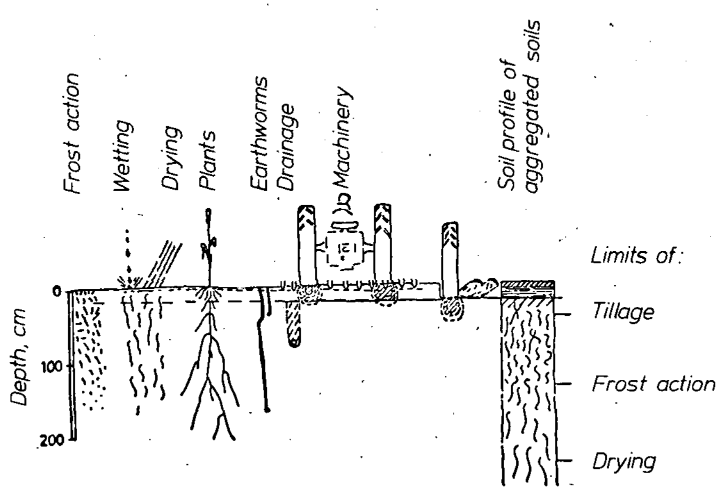

Fig.3. A: Some important factors for profile formation in cultivated soils. The macrostructure pattern in aggregated soil (clay soil) with approximate structural boundaries.

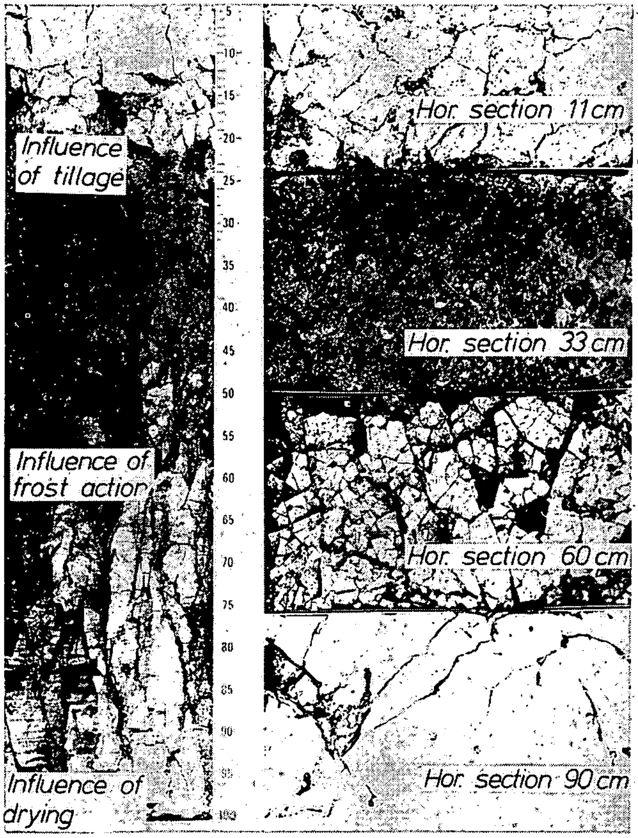

Fig. 3 B: *Diagram of soil profile (Ultuna 4-55, Andersson and Wiklert) showing the pattern of the macrostructure in one vertical plane and four horizontal planes.*

There is a great variation in the depth functions of the permeability from soil to soil type but some generalization can be done. Well aggregated clay soils often have a high permeability dow to 50-70 cm, it diminishes in the layers down to 120 cm to one-tenth and to very low values in the bottom to 200 cm depth.

Table 1. Drain test field No.51 to 62 observed in a number of years in the period 1947-1977. Clay content in subsoil, drainable pore space, hydraulic conductivity: k_v = vertical with the core method, k_a = augerhole method, the drain method, the drain depth used and finally optimum distance found.

Test field	Number of years observed	Clay content in subsoil	Drain-pore space	Hydraulic conductivity (m/day) in the horizons				Drain depth used	Drain spacing optim.
				20-50 cm	50-100 cm	50-100 cm	100-180 cm		
No.		%	μ	k_v	k_v	k_a	k_a	m	m
51	17	55	6.1	2.5	0.2	0.15	0.10	0.85	16
52	20	60	4.1	3.4	2.3	0.28	0.28	1.00	16
53	10	50	7.5	4.4	0.16	0.30	0.02	0.8	20
54	19	60	-	2.3	0.21	0.06	0.19	0.95	16
55	14	70	6.3	1.7	1.2	0.05	0.05	0.85	16
56	17	17	9.8	0.28	0.37	-	0.3	0.9	25-30
57	11	40	-	1.92	0.06	0.04	0.01	0.8	12-16
58	20	60	5.6	0.46	0.01	0.01	0.00	0.75	12-16
59	5	50	-	3.80	0.03	0.01	-	0.8	12-16
60	15	82	4.7	0.11	0.04	0.06	0.04	0.65	14-16
61	19	47	4.7	2.21	0.04	0.06	0.06	0.9	18
62	21	70	2.2	0.01	0.02	0.04	0.02	0.7-1.0	16

Direct and indirect drainage effects

How the groundwater level varies and how far it is influenced by drainage is shown in Fig.4. In years with normal rainfall (1952) one can tell about a crop season and an off season period. In a year with high rainfall or an uneven distribution as in 1953, the drains have to work even in the crop season. During the early spring the groundwater level rises in connection with the melting away of the snow and the thawing of the soil. During the late spring the drainable water will flow out in natural or artificial drainage and the groundwater level will normally go down to the pipe level as shown in the groundwater diagram.

When the crops start growing their water consumption will influence the groundwater conditions to a high degree. Water is taken out by the roots even below the pipe level and the groundwater level will also drop below. During autumn with increasing precipitation and decreasing evapotranspiration the water level rises above the pipe level.

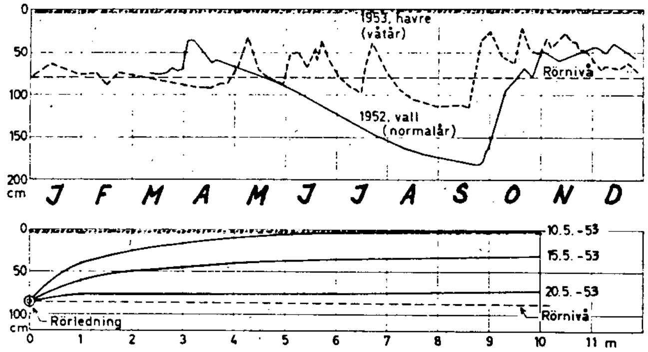

Fig.4. The variation of the groundwater level in a clay profile with about 50% clay and a developed structure to 2 m. The water conductivity 0.2-0.4 m/day and a drainable pore space of 6%. Drain test field Lanna, Skaraborgs county, 20 m drain space, 0.9 m drain depth. The upper diagram shows the average groundwater level in the normal year 1952 with a grass crop and the wet year 1953 with oats. The lower diagram shows the forms and the drawdown of water level in a 10-day period.

Due to the properties of the soil profile, especially the permeability, the drainage effect will diminish more or less rapid with the distance from a drain. The groundwater level in the vincinity of the drain will be kept down deeper which will empty the macropore system more effectively. This primary effect of a drain will induce several secondary effects of physical and bio-chemical nature which are finally synthesised in the plant growth, in the workability of the surface layer and in the variation of the trafficability.

In Fig.5, as one example, the influence of drainage on plant growth is shown from the drain test field No.52, Gunnarstorp, Skaraborgs county. The soil has 60% clay with a drainable pore space of 4%. In this particular crop season (1966) an average high groundwater level gave a bad workability in the spring, a coarse structure in the seedbed, a compressed central part of the topsoil with low permeability and finally in the crop season high water saturation in the topsoil. A high depression in the growth of the oat plants can be seen on the 80 m space. Even on the 24 and 32 m a depression in the height and the yield could be found. On 16 m the growth conditions were acceptable over the space.

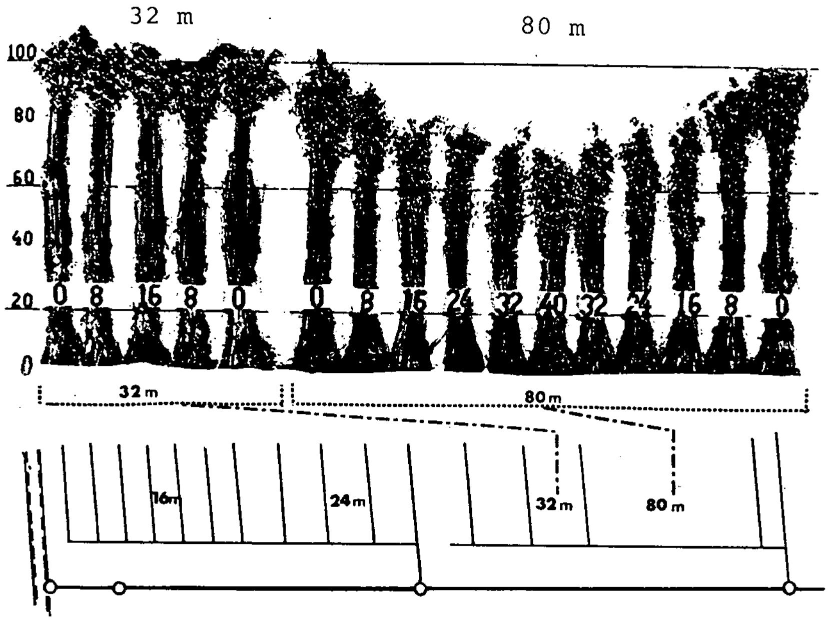

Fig.5. The influence of drainage in a spring sown, cereal crop, oats. From the drain test field No.52, Gunnarstorp, Skaraborgs county. In the test field the distances 16, 24, 32 and 80 m are checked. The picture shows a view of the crop stand in September 1966 on some points from the drains on 32 m and on 80 m.

The variation in yield shown by this example has been established through harvesting of test plots continuously from drain to drain on all test fields. Out of this data the correlations between drain space and yield have been calculated. Fig.6 shows the general fact that winter crops, because of more frequent off-season effects, give a higher response to drainage than spring crops. The off season effects are often a combination of high topsoil saturation, frost heaving, plant pathogens, etc. (Håkansson, 1960, 1961).

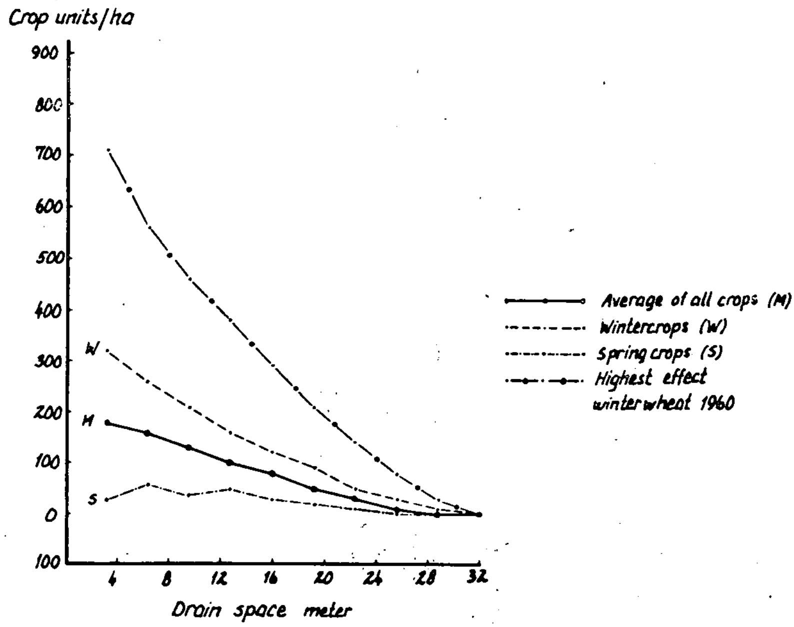

Fig.6. The drain test field No.52, Gunnarstorp. Correlation between drain space and yield. The curves are moved to the same zero level and indicate the variation in yield when the drain space is diminished from 32 m.

Drain intensity

The choice of space in the design of drainage is based on the results and experience gained from the test fields. One compares the cost of drainage for a higher drain intensity with the value of the yield increase. The value of better workability and trafficability is thereafter weighed in. For separate agricultural regions certain average intensities will be applied according to crop rotation and demand of workability and trafficability.

The drain space is in practice varied in the interval 10-30 m (Fig.7). The total length of the laterals per ha will in proportion vary between 1000 m to about 300 m. The length per ha of the mains needed can vary between 80 and 120 m. In the diagram Fig.7 an average of 100 m has been calculated with.

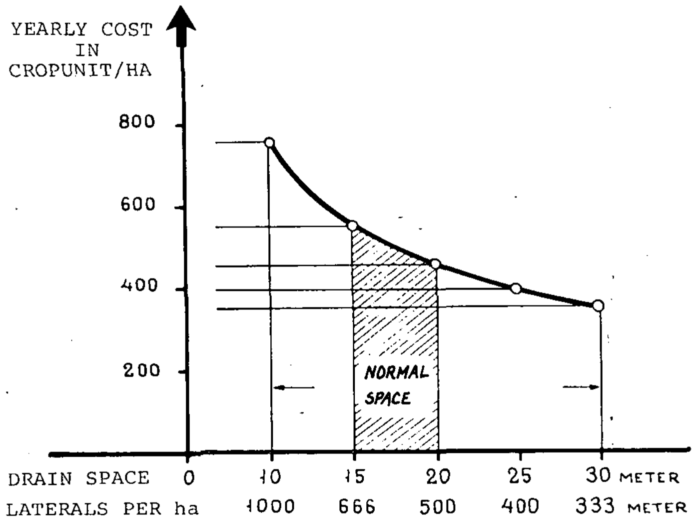

Fig. 7. Yearly cost for drainage with a variation of the drain space from 10 to 30 m. The drain space is most frequently chosen in the interval 15-20 m.

In the calculations of the yearly cost, a time for discounting of the investment cost of 30 years, 7.5 per cent interest and a recurrent cost of 0.5 per cent of the investment cost has been assumed. The yearly cost has been transformed into crop unit/ha. The crop unit has been given a value of 0.50 kronor.

If the drainage cost is compared with the benefits one can in normal case find a drain space in the interval 15-20 m justified. Some hard to drain soils, as heavy clay soils or soils with frost heaving or low bearing capacity, have shown in the drain test program that a drain space in the interval 10-15 cm can pay. In fields with a high natural drainage or in an area with low rainfall the intensity can be chosen in the interval 20-30 m.

The depth of the laterals in a drainage system is normally set at 80-90 cm. The maximum in permeability which normally lies in the niveau 30-80 cm in the profile is thereby utilized. The drain tests have, however, shown that on many soils a bigger depth gives higher yield. The average rise of yield is about 100 kg per 10 cm deeper drainage between 0.8-1.2 m. The value of this yield, however, just outweighs the cost for deeper drainage (Håkansson,1969).

2. The root growth and the root environment

Root types and root systems

A plant-root system is often just as dependent on inherited traits and just as set in form and characteristics as the above-surface plant parts. Nevertheless, growing conditions modify to a large degree the root system and provide the framework for its expansion. A plant can develop a deep and well-branched root system in a well-structured and well-drained soil.Another plant of the same variety will have a shallow root system with a different branching pattern in a single-grain soil, in a soil with a high water table, or in a soil with a dense horizon. The whole root system of dicotyledonous plants (herbs) is formed by one main root from which lateral roots grow. On grasses several (3-5) equal-sized roots emerge one after the other at time of germination. Besides these seminal or primary roots, so-called crown roots, playing a more or less important role in the water and nutrient supply, develop from the basal parts of the stem.

Growth mechanisms of root tips and root hairs

The roots, and primarily the root tips, are very effective organs for water and nutrient uptake. The intensity of root branching results in hundreds of thousands of root tips on a mature plant. The number of root tips is truly the most important aspect in the plant's ability to take up water and nutrients. At the very tip of the root, the root cap (Fig.8) protects the meristematic region where new cells are being produced by cell division. These cells later enlarge in the zone of elongation. This cell elongation causes the meristematic region with the root cap to be pushed forward, resulting in the longitudinal growth of the root. Only a small part of the root, at most a few millimeters, is pushed through the soil. As the root tip is pushed forward, the outer cells of the root cap are sluffed off but are replaced by new growth at the root apex.

After cell elongation and the consequent forward movement of the root surface ceases, root hairs can develop. Root hairs are hair-like growths from the epidermal cells of the roots and are about 0.01 mm in diameter and 1 to 10 mm in length. The root-hair zone varies from a few millimeters to several centimeters long depending on plant variety and conditions under which the roots develop. On every rapidly growing root tip, new root hairs are continually formed which, during their development, reach new parts of the soil.

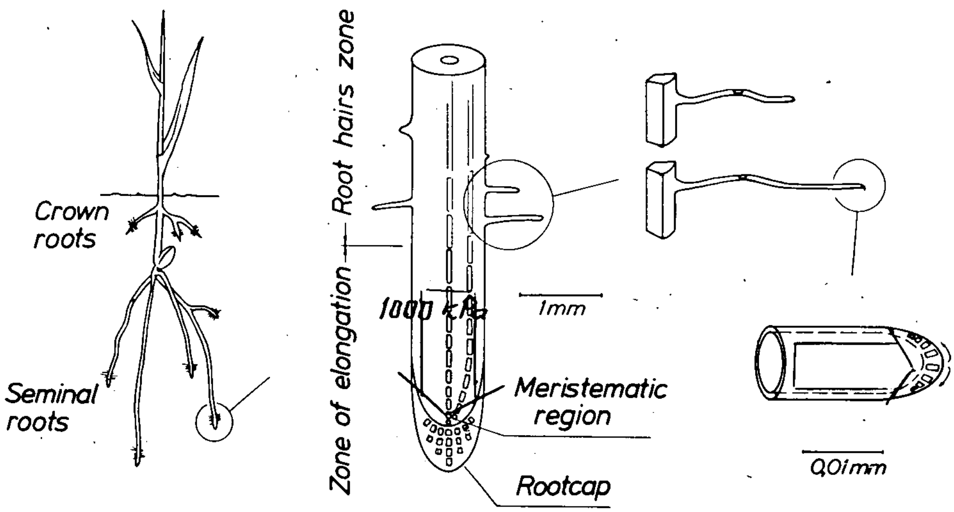

Fig.8. A sketch of the root system of small grains. Structure and growth mechanism of the root tip and root hair.

The effective root absorption surface is increased very much by the presence of root hairs. A wheat root of 0.5 mm diameter can have a surface absorption area of 5 cm^2 per cm root length. The growth mechanism of root hairs is similar to that of the root, and growth occurs by accumulation of materials in the tip of the root hair. This type of growth enables the root hairs to penetrate complicated pore channels among and in the aggregates. The new root surface secretes slime, further increasing contact with the soil particles.

Growth rate of roots

Primary roots of small grains will, for example, in soils with favourable structure, grow at the rate of 0.5 to 3.0 cm/day and will grow to a length of 1 to 2 m. The growth rate of lateral roots is slower than that of primary roots, and their final length less. Frequency of branching commonly varies between 0.5 and 5 roots per cm of the parent root. According to investigations by Wiklert (1969), root branching begins 15 to 35 cm behind the tip.

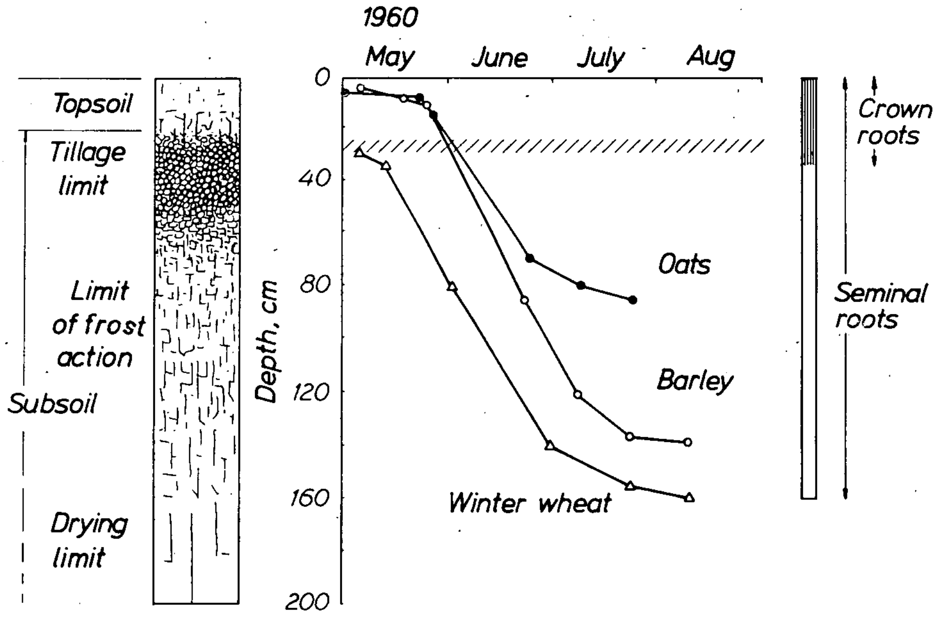

Fig.9. Development of the root systems of winter wheat, barley and oats in aggregated soil (according to Wiklert).

Depth penetration of a root system in aggregated soil is illustrated according to Wiklert's data in Fig.9. Root penetration relationships among the three grain varieties (winter wheat, barley, and oats) are, for similar root environments, most commonly as shown in the figure. Oats are not particularly deep rooted; the individual roots are vigorous and well branched. The barley roots are less vigorous and appear weaker. The barley root system is similar to oats in branching frequency but goes deeper. The crown roots of small grains generally are limited to the topsoil or slightly deeper. According to this investigation, rate of depth penetration for the various cultivars was 2-3 cm per day during the most rapid growth period which ends at time of heading. A rapid growth rate like this is only possible by completely unimpeded growth of root tips in an open pore system or in a system of soil cracks that gradually open as the roots remove water.

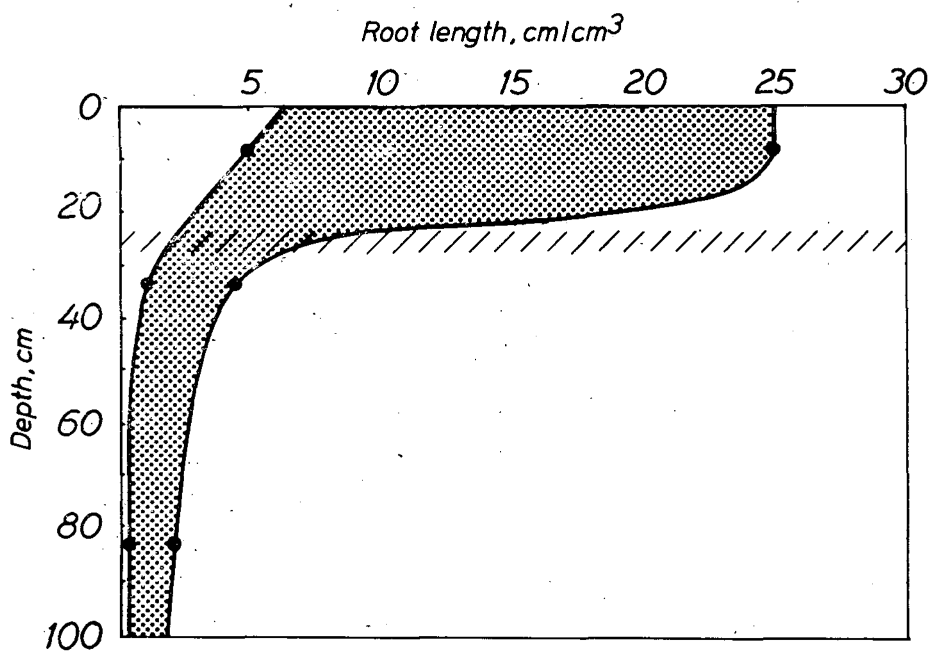

Fig.10. Root frequency for small grains in terms of root length per cm³ of soil (according to Barley, 1970).

To evaluate the effectiveness of different root systems, data about root frequency in different horizons of the soil is also of interest. In this respect, root length per volume of soil is most informative but data of that type are very few. Root length in cm per cm^3 for different grain varieties is, according to foreign investigations, given in Fig.10. The average root length in the soil profile under a fully developed crop will usually be:

in the sopsoil, 10 cm/cm^3; at a 0.5 m depth, 1 cm/cm^3; and at a 1.0 m depth, 0.1 cm/cm^3. A commonly reported average value for deep-rooted crops is 1 cm per cm^3 at 1 m depth. Even less information is available about root length per unit area of soil surface. Values ranging from 50 to 500 cm per cm^2 of soil surface have been reported. According to the cited investigations of the root system distribution in the soil profile, it seems very important to have a good root environment in the topsoil and in the top portion of the soil profile; a slight change of structure in these portions of the soil can be very detrimental. It is primarily there that heavy vehicle traffic affects the pore system and consequently, the root environment.

3. Compaction-induced changes in soil properties

Compaction pressure - pore space distribution

The primary effect of compaction on the soil is to decrease total pore volume and the coarse pores are the first to be affected. The geometry of the pore system is extremely complicated because pores continuously change in size and shape along their length. Therefore, it is not possible to give an exact description of a pore. Instead, it is common to characterize a pore with an equivalent diameter based on the suction, i.e. the soil water pressure that has to develop to empty a pore of water. This so-called equivalent pore diameter, d_v, in cm can be obtained from the equation $d_v = 0.3/h_t$ where h_t is the soil water pressure in terms of the height in cm of a water column. A tile drain at 1 m depth develops a soil water pressure of 100 cm in the pore system of the topsoil. Thus, the pores that will empty in this case, can be described with the help of the equation $d_v = 0.3/h_t = 0.3/100 = 0.003$ cm = 0.03 mm.

Soil water tension can vary from 0 at saturation to about 100,000 m water-column equivalent under very dry conditions. Nature can be simulated experimentally with the aid of the pressure chamber technique and the humidity chamber technique and in this way the pore system and its water retention capacity can be described. In a discussion of soil-water relationships in connection with soil tillage, drainage, and irrigation, the above mentioned concepts are basic and will be considered more and more in the future.

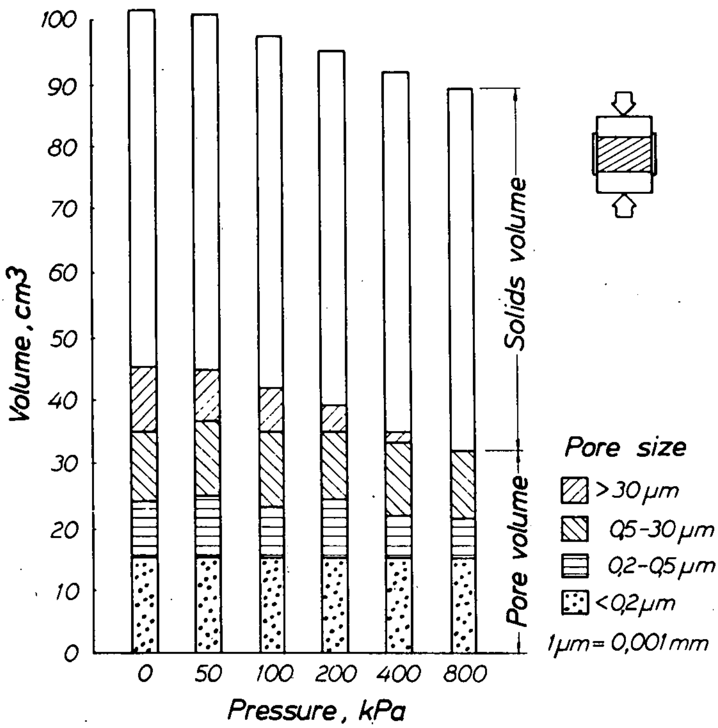

Fig. 11. Changes in pore volume and pore size distribution with increasing pressure. Grävsta, Uppsala county, 40.0 to 42.5 cm horizon, well aggregated clay soil, clay content 40%.

The results of an investigation by the Department of Agricultural Hydrotechnics on undisturbed clay soil in Grävsta, Uppsala county are shown in Fig.11. Undisturbed cores were taken at a depth of 40 to 42.5 cm. The cores were maintained at a soil water tension of 0.05 m and were successively exposed to pressure increments from 0 to 800 kPa (0-8.0 kg/cm^2). Pore volume and pore-size distribution were determined after each pressure increase. In undisturbed condition, this clay soil (clay content, 40%) has a very favourable structure with about 10% of the pores larger than 0.03 mm, which drain at a tension of 1 m. It is these pores that provide ready access for roots and secure high water permeability. It is noticed (Fig.11), that the number of coarse pores decrease successively with increasing pressure. The coarse pores are completely compressed at 800 kPa (8.0 kg/cm^2). Already at a pressure of 200 kPa, the number of coarse pores begin to reach critically small values. Contrarily, one cay say that the coarse pore system has a structure that can withstand a maximum pressure of approximately this value.

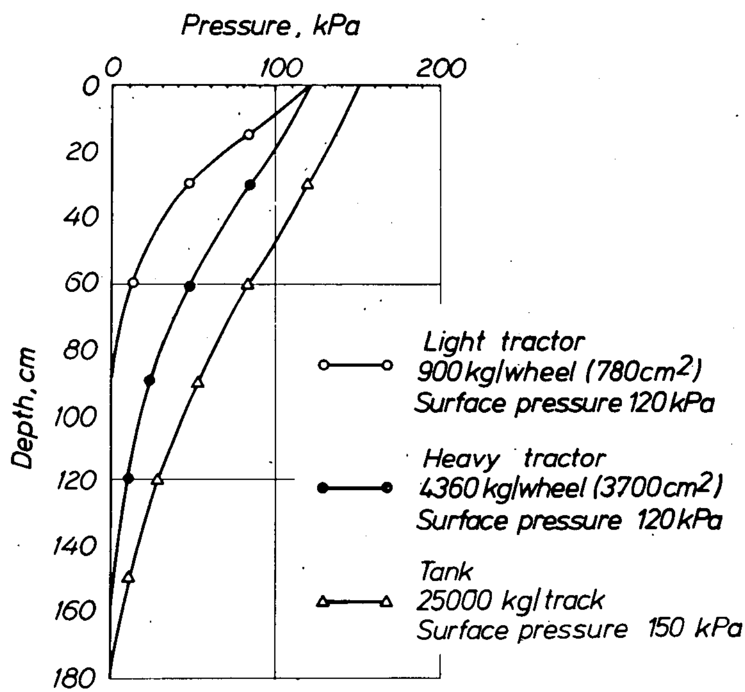

Fig.12. Increase in soil pressure under a light tractor with 0.9-ton wheel-weight and tyre width of 30 cm; heavy tractor with 4.4-ton wheel weight and tyre width of 62 cm; battle tank with total weight of 50 tons and track width of 61 cm.

As the coarse pores collapse, smaller pores are formed. The pores smaller than 0.03 mm were, therefore, maintained until a pressure of 200 kPa (2.0 kg/cm^2) was reached. At pressures beyond 200 kPa, this portion of the pore system was, however, also affected. The same relationships that are described in this detailed example of an aggregated clay soil have been observed in studies of a series of fields with different soil types and pore volume distributions. The coarse pores begin to break down at a pressure of about 200 kPa.

The pressure exerted by a vehicle load at the soil surface diminishes with depth in the profile. As an example of what effect extremely heavy vehicle traffic has on the soil, we shall herein describe the results of an investigation where two identical clay soil profiles at a military exercise area were compared. One profile has been exposed to heavy military traffic for 30 years, of which 15 years with 50-ton tanks, the other has been protected and in grass during the same period; previously the entire research area was cultivated. Maximum pressures that occurred in the profile are shown in Fig.12. For purposes of comparison, calculated pressure curves from light and heavy tractors are included. In the top portions of the profile, pressures from the tanks are comparable to those from agricultural transport vehicles while in the deeper portions, they are substantially higher.

The comparison of compacted and uncompacted profiles, to 1 m depth, includes structure, earthworm-hole frequency, macro-aggregation, water infiltration, total pore volume, pores larger than 0.03 mm, and shrinkage. A clear difference could be established between profiles in all properties investigated. The clearest connection between pressure and physical changes was found in total pore volume and in the coarse pore system > 0.03 mm. The effects could be detected to the 100 cm depth (Fig.13a and b). In the top 50 cm of the profile, the influence on macrostructure and earthworm-hole could be visually observed. There was also a clear difference in permeability, a property markedly dependent on the macropore system, between the profiles in the top 50 cm. The influences were less below 50 cm (Fig.13c).

Compaction pressure - air permeability

The soil air is the source of oxygen needed for root respiration and the oxygen demand of microbial organisms. On the other hand, it is the recipient of the carbon dioxide produced by root respiration and by the microbial organisms. In order for the soil air not to be depleted of oxygen and be filled

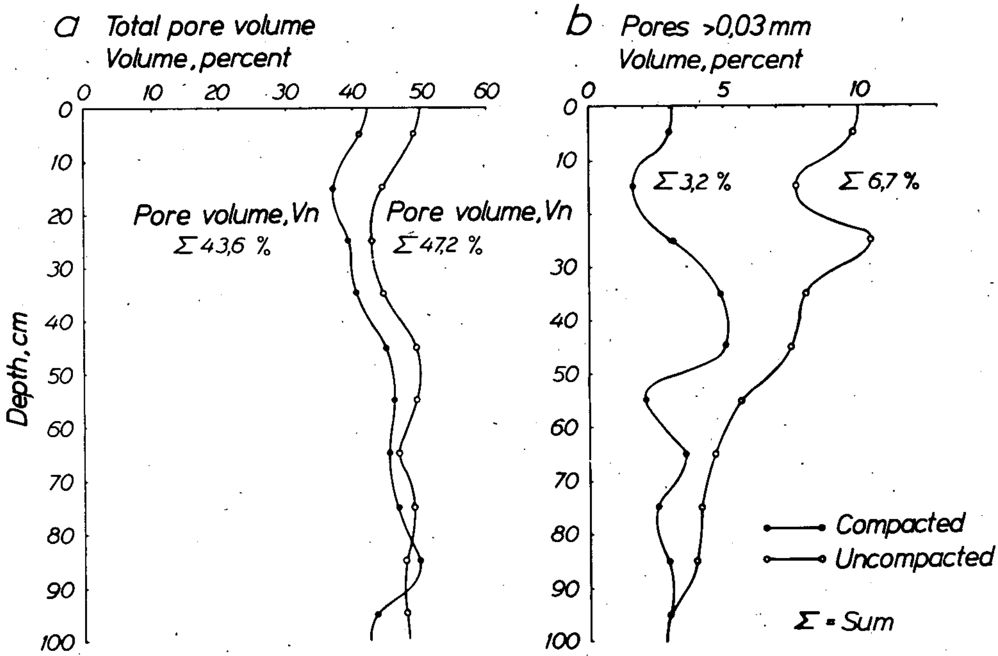

Fig.13. a) Comparison of total pore volume in a protected soil profile below a grassed surface, with a soil profile exposed to heavy traffic during 30 years, 15 of which were with 50-ton battle tanks. b) Comparisons of number of pores greater than 0.03 mm (drainable at 1 m tension).

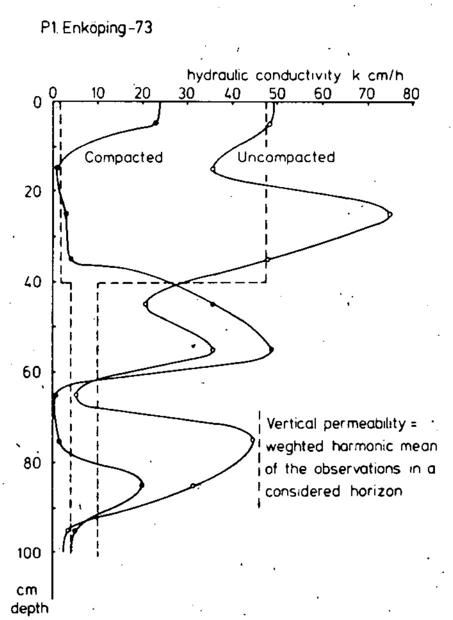

Fig.13 c) Hydraulic conductivity, k cm/h, to a depth of 100 cm in the compacted and uncompacted profiles. The vertical permeability 0-40 cm and 40-100 cm calculated as weighted harmonic mean of 10 cm horizons.

with carbon dioxide, it must be exchanged. Gas transport can be accomplished partly through diffusion, i.e. molecular movements in the stagnant air mass, and partly through mass transport, i.e. streaming of the whole air mass. For these air movements, the coarse pore system which can be drained at small tensions is of great importance. Diffusion and streaming depend on the air-filled pore volume; in addition, streaming depends on a certain number of coarse pores with low streaming resistance. When drainage is poor, air exchange is completely dependent on the coarse pores because the small pores are blocked by water. Oxygen and carbon dioxide differences in the soil are, however, balanced mainly through the process called *diffusion*.

The rate of respiration is determined by the mass and vigor of roots and microbes plus the soil temperature. The respiration process is considered to consume 5 to 20 liters oxygen and to produce an equivalent amount of carbon dioxide per 24 hours per square meter of soil surface.

A complete infiltration of the soil profile with roots is dependent on a well-developed network of cracks and channels for the movement of air, permitting the roots to push forward in all directions in close contact with the soil-air's supply of oxygen as well as the soil's supply of water and nutrients. Roots cannot commonly penetrate more than a few millimeters into free soil water from the boundary between air and water. It has been shown experimentally that roots, to maintain normal growth, need to have close contact with air containing at least 8 to 10% oxygen. Commonly, this oxygen concentration can be maintained as long as the air-filled pore volume does not drop below 5 to 10%. The active root is always surrounded by a water film through which the oxygen must diffuse. It has been shown experimentally that this water film will attain a thickness that permits optimum oxygen supply and root growth at a soil water pressure equivalent to 1 m suction.

That part of the pore system which is free of water determines the air exchange in the soil, regardless of whether it takes place in the form of diffusion or streaming. Therefore, a simple air permeability determination is a good measure of the air exchange capacity. It is most convenient to perform this determination at a suction of between 1 and 6 m.

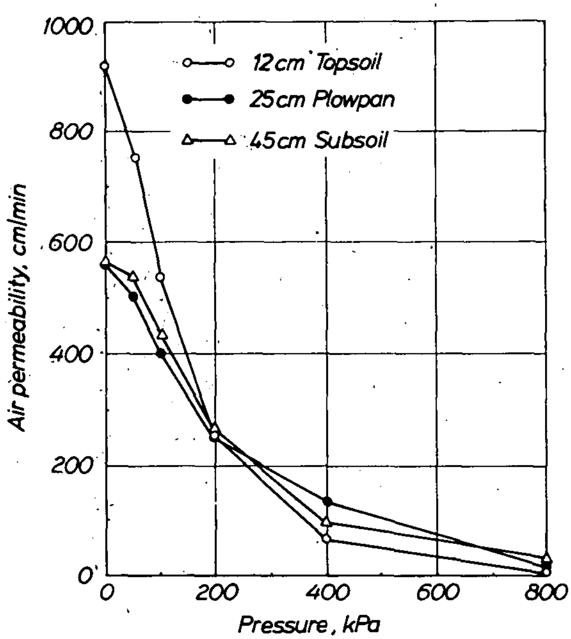

Fig. 14. Relationship between pressure increase and air permeability. Topsoil, plough layer and subsoil in claysoil profile, Ultuna.

The relationship between compaction pressure and air permeability for clay soil, Ultuna D-1973, is shown in Fig. 14. Cores were taken from topsoil, ploughsole, and subsoil and drained at 6 m equivalent water tension. Subsequently, they were exposed to pressures of 50, 100, 200, 400 and 800 kPa and air permeability was determined. As seen in the figure, air permeability decreased greatly with pressures even as low as 200 kPa and reached very low values at 800 kPa. The test site, with a third-year hay crop, had a relatively uniform topsoil and air permeability was initially high, indicating the presence of coarse pores. However, the coarse pores in the topsoil apparently collapsed more easily with increasing pressure, and air permeability became lower than that of the ploughsole and subsoil at 400 and 800 kPa. In agreement with results from the investigation of pressure increase and pore-size distributions, it can be concluded that air exchange gradually deteriorates with increasing pressure and becomes critically small at pressures above 200 kPa. This is especially true at suctions of 1 m because more pores are blocked by water, with a consequent considerable reduction permeability, as compared to 6 m suction.

Compaction pressure - penetration resistance

According to several investigations, a root tip can develop a maximum pressure of 1,000 kPa. The larger cracks and channels provide unrestricted room for root growth. As pore diameter approaches that of root diameter, pore tortuosity will influence root growth. Roots cannot penetrate a completely rigid matrix unless the pore diameter is the same, or larger than that of the root diameter. Soil mechanical resistance is closely related to soil density and texture. Depending on soil type, root growth is halted at a volume weight somewhere between 1.3 and 1.8 kg/dm^3. Attempts have also been made to relate root growth directly to the mechanical properties of soils, for example, penetration resistance. However, soil resistance to a root tip compared to a steel tip of the same dimension, is considerably lower. The needed force for root penetration in an average soil has been found to be 1/4 that needed for penetration of a steel penetrometer. In a compacted soil the relative power need is only 1/8. In soils suitable for root penetration, limiting values ranging between 800 and 5,000 kPa have been obtained with steel penetrometers. Several explanations can be given for the ability of the root tip to penetrate so easily.

a) The root tip, in contrast to the steel tip, follows the path of least resistance and takes advantage of small variations in density since it has a certain amount of freedom of movement.

b) The root tip has a tendency to compress the soil in a cylindrical fashion, i.e. the forces developed by the tip have a relatively large component at right angles to the direction of growth and relatively small components in the direction of growth.

c) When the root encounters resistance, it will, just behind the root cap, expand two to three times its normal diameter; this radial expansion of the root causes the soil resistance to be eased against the root tip itself so that it can further penetrate, as illustrated in Fig.15.

d) Fluid transport to and from the root tip can alter the mechanical properties of a soil; water uptake by the root causes shrinkage cracks, and cracks originally inaccessible to the root open up.

e) Friction against the root tip is considered to be small, partly because of its ability, even at high soil resistances, to maintain a pointed form and partly because it secretes plenty of slime.

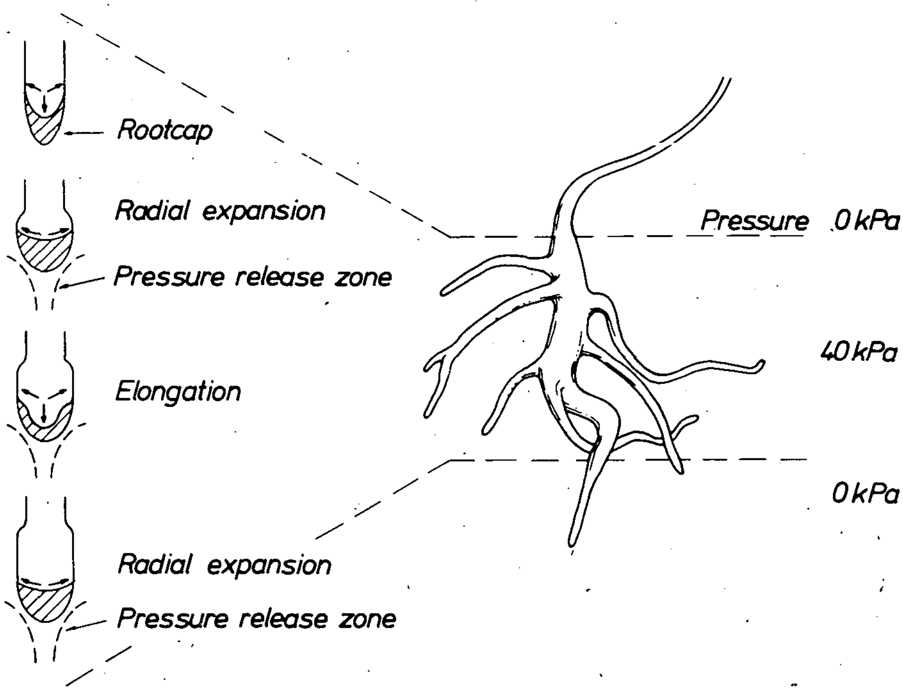

Fig. 15. Reaction of roots to high soil resistance. There is partly an increase in size of radius resulting in a pressure decrease in front of the root tip, and partly a profuse branching allowing some branches to find paths of less resistance (after Abdulla et al., 1969).

The relationship between pressure increases and penetration resistance in a clay soil from Ultuna is illustrated in Fig.16. Penetration measurements were accomplished using 2 mm diameter needles with 60° cones. Each value is the average of twenty-four measurements and is shown in kPa. The penetration resistance was measured at suction of 0.05 and 6.0 m. At a soil suction of 0.05 m under non-compacted conditions (0-treatment), the resistance of all investigated horizons was under 1,000 kPa. With increases in compaction pressures up to 200 kPa, the resistance increased quite sharply. The mechanical properties of the topsoil especially seem to change in this regard, but the penetration resistance of the ploughsole also increased sharply. The subsoil, which had the most favourable structure, changed the least. The sharp increase in penetration resistance that occurred when the soil suction was raised from 0.05 to 6 m is also interesting. All curves at the 6 m soil suction lie above the values of penetration resistance, that, according to the earlier mentioned investigations, can be judged to give a reasonable root resistance. A 200 kPa compaction pressure results in a penetration resistance that, according to the following study about root growth, results in a critically limiting value for root penetration.

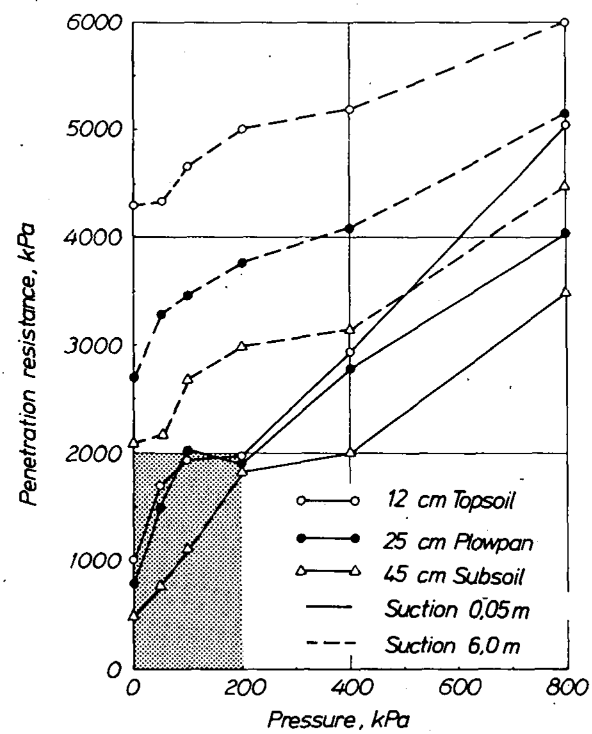

Fig. 16. Relationship between pressure increases and penetration resistance at 0.05 m and 6.0 m tensions. Topsoil, plough layer and subsoil in clay-soil profile at Ultuna. Shaded portion indicates an area of moderate root resistance.

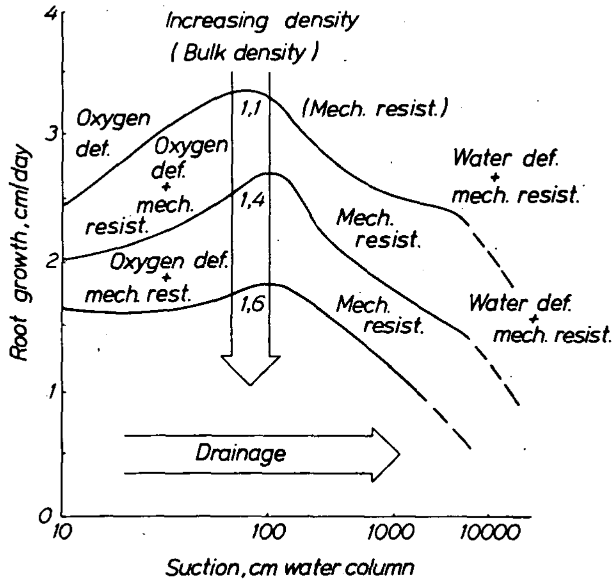

Fig.17. Root growth at different degress of compaction and soil tension. Soil type: loamy sand (according to Eavis, 1972).

Compaction pressure - root growth

Under field conditions, the soil is continuously exposed to internal and external forces that tend to change the soil's total volume. When a soil is compressed, the pore volume decreases, bulk density increases, and the proportion of large pores decreases. The changes in the soil volume can be caused by shrinking and swelling of the soil matrix which in turn is caused by freezing and thawing or alternate wetting and drying, and also from compaction pressures in the soil profile by machines, vehicles and implements.

The physical condition of the soil influences the amount of water and nutrients that the root system takes up. This influence can arise from changes in the soil's ability to store and conduct water and nutrients or indirectly from effects on root growth and root functions and also from chemical and biological reactions in the soil. The interactions among root growth, mechanical resistance, oxygen availability, and water availability are apparent from Fig.17 (Eavis, 1972). Root growth of field peas with increasing suction in soil (loamy sand) of various densities is illustrated in the figure. With increased degree of compaction - increased bulk density - root growth decreases. There is a fairly wide maximum range in root growth at about 100 cm tension. Root growth is retarded at tensions below 100 cm because of oxygen deficiency. At high tensions, the firmness of the soil matrix increases and root growth decreases because of mechanical resistance. With further increases in tension, water deficiency also becomes a cause of reduced root growth. Mechanical resistance becomes more apparent at higher densities.

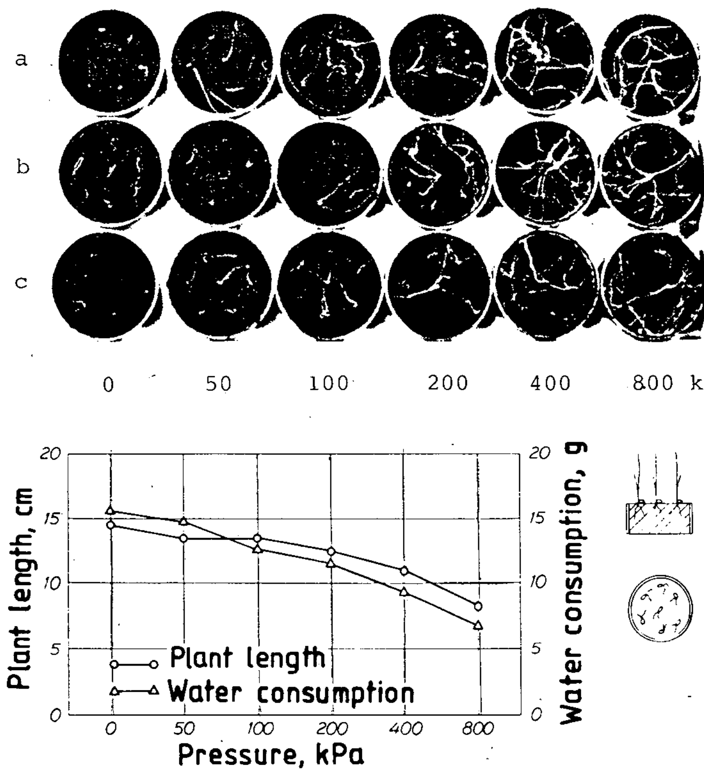

Fig. 18. Relationships between compaction pressure, plant height, root development and water use for spring wheat. The wheat was grown on soil cores as shown in the diagram. The photograph of the upper surface of the cores shows an increasing number of roots on the surface, i.e. less root penetration with increasing degree of compaction.

a) Topsoil. b) Plough layer. c) Subsoil.

Clay-soil profile, Ultuna.

The relationship among compaction pressures, root growth, water use, and plant length in a clay soil is illustrated in Fig.18. Wheat was grown on soil cores which were 72 mm in diameter, 25 mm in height, and packed with pressures up to 800 pKa. Soil samples from the topsoil, ploughsole, and subsoil were taken from a well-aggregated clay soil at Ultuna (profile D72). Soil suction was 0.05 m. Seven wheat kernels were placed on the surface of each soil core and allowed to grow and develop roots. As is apparent from the picture of the roots, their penetration was unrestricted through the surface of the control treatment (0 kPa). At pressures of 50 and 100 kPa, root penetration was likewise good though a few roots had a tendency to grow laterally on the surface of the soil core. At 200 kPa, roots began to experience more restriction to penetration and grew for long distances on the surface, particularly in the case of the ploughsole. At 400 kPa, root penetration was poor and at 800 kPa, it was completely stopped and all roots grew on the surface. There was good wheat growth on the control treatment and at the lower pressures where the roots penetrated the core. At 200 kPa, plant growth began to decrease and at 400 and 800 kPa there was a further decrease. Water use also decreased with increased compaction. Wheat plants at 400 and 800 kPa were first to show water stress and to wilt. The experiment ended at this stage. Both the root pictures and curves depicting plant length and water use suggest a sequential influence of pressure increases on the root environment, and that when a pressure of 200 kPa is reached, a relatively large negative influence is imparted. Root penetration is brought to a complete halt by mechanical resistance at the high compaction pressures. Thus, again, 200 kPa emerges as a critical value.

Drainage - compaction vulnerability

The vulnerability of a soil to compaction decreases with increasing suction or degree of dryness. Tile drains will create a maximum soil-water pressure equivalent to a 1 m water column. At that water content, the soil is very vulnerable to compaction. More water must be removed by evaporation before the soil is suitable for cultivation and, according to laboratory experiments, the soil is then at a stage of dryness equivalent to a soil water pressure of 6.0 to 60.0 m.

Coarse-textured soils lie near the lower limit of this range, while clay soils don't gain necessary firmness, and cannot be cultivated without compaction damage occurring until a pressure equivalent of 60.0 m is reached. In a study of compaction on a series of clay soils with clay content between 40 and 80%, it was found, among other things that compression from a compaction of 200 kPa at 6 m suction repeatedly averaged 80% of the compression resulting from the same compaction pressure at a high degree of saturation, i.e. 0.05 m suction.

On areas with tile drains, a rapid removal of surplus water is insured and the water table drops to the level of the drainage tiles, i.e. at the most 1 m. By direct evaporation from the surface, the topsoil eventually reaches a degree of dryness that will be within the stated range of 6 to 60 m and the soil can be cultivated and seeded.

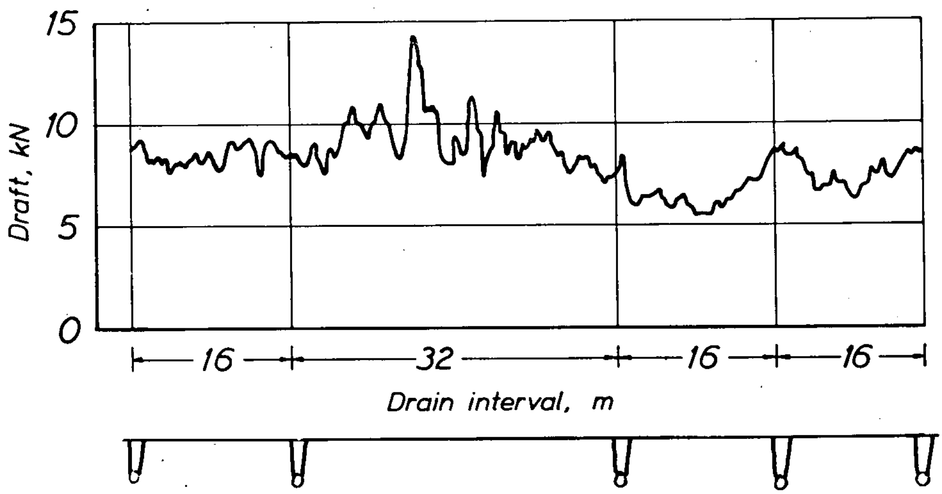

Fig. 19. Results from a combined drainage - time of seeding - experiment at Lanna experimental farm, Skaraborg county. Variations in draft requirements in intervals between covered drains at two spacings, 16 m and 32 m. Measurement 1967-09-13 during fall ploughing. The topsoil was dry.

Results from a combined drainage and time-of-seeding experiment, conducted at Lanna Research Farm, Västergötland, are shown in Fig.19. Drain intervals of 16, 32 and 80 m were included in the experiment; depth to the tile was 0.8 m. A clear difference in soil dryness among tile distances is most often evident during the spring, and the 16 m interval can be used during the normal period for spring tillage without incurring any compaction damage. This was the case during spring tillage in 1967 for this experiment. In contrast, at normal time for spring tillage, the 32 m, and especially the 80 m interval, was so wet that severe compaction and unfavourable structure in the topsoil resulted. This was reflected in crop development, in final yields, and in the resistance of the central part of the topsoil to fall ploughing. Draft requirements were measured in conjunction with the harvest when the soil was dry, and a portion of measurements for the 16 m and 32 m tile drain spacings are illustrated in Fig.19. At a tile drain spacing of 16 m the soil structure was good and the drag resistance was low and even, while, at the 32 m distance, compaction during spring tillage resulted in an unfavourable structure difficult to penetrate with a ploughshare. The soil broke into large clumps and the draft requirement increased stepwise to double that of 16 m. Soil water content is one of the most important factors determining compaction vulnerability and structural deterioration. The most important possibility for regulating water content is by drainage, which is nicely illustrated by the results of this experiment. There are available data over a 25-year period pertaining to the relationship between tile spacing and dryness of Swedish cultivated soils that, in general, confirm this conclusion.

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Paper 1.10

AN ELECTRIC ANALOG FOR UNSATURATED FLOW AND ACCUMULATION OF MOISTURE IN SOILS

(This workshop paper will be published in one of the forthcoming issues of the Journal of Hydrology. Therefore, these proceedings contain only the abstract.)

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Abstract

An electronic analog model of the unsaturated zone was developed. The similarity between the integrated flow equation and Ohm's law is the base of the model. The main difference between the two equations is compensated for by amplifiers. The model simulates one day in 2 seconds. There are ten normal layers, each with adjustable magnitude. Moreover there is a top layer in which infiltration, ponding and run-off are simulated, and a drain layer with adjustable drain-intensity. The normal layers are containing an adjustable resistor for the connection with an other layer and a function generator for the $K(\theta)$ relation. There are two transition layers which have to be placed at the boundary between two layers of different soil properties.

In saturated conditions the model is acting incorrectly. This causes a calculated thickness of the saturated layers of a too small magnitude, which can be compensated for by using an equivalent drain intensity, lower than the real one.

This model can be used to simulate the effect of natural rainfall and evaporation on moisture content at every depth. Soil physical properties and drainage conditions can be adjusted. Homogeneous as well as layered soil can be represented by the model. It can especially be applied to investigate drainage requirements of soils.

Some examples are given to show the applicability of the model. A short technical description is given at the end.

Paper 1.11

SIMULATION OVER 35 YEARS OF THE MOISTURE CONTENT OF A TOPSOIL WITH AN ELECTRIC ANALOG FOR 3 DRAIN DEPTHS AND 3 DRAIN SPACINGS

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Summary

With an electronic analog model the moisture content of the top 10 cm of a schematized silt loam soil was calculated over 35 years. Drain depths of 70, 100 and 130 cm below surface were used, each with three drainage intensities as shown in Table 1.

To obtain a low number of days with a very wet soil, a high drainage intensity appeared to be important, the drainage, however, should not be shallow.

To obtain a high number of workable days deep drainage is important, and drainage intensity has little significance.

Introduction

Drain depth and drainage intensity both affect the moisture content of topsoils. The Netherlands drainage criterion pays more attention to drainage intensity than to drain depth. It was found by WIND (1976) that as regards workability, drain depth is more important than drainage intensity. Because of this difference in point of view we were anxious to see what interrelations exist between depth and intensity.

The electronic analog (WIND and MAZEE, 1978) gives the possibility to simulate long time-series with low costs. Therefore we simulated 35 years between 1941 and 1977 from September 1 to May 31, for three drain depths and three drainage intensities. So about 85,000 days were calculated, which calculation lasted about 48 hours. The same simulation executed with a numerical model would have cost US $ 80,000.

The aim was to determine during how many days there was:

1. a very wet soil with less than 2% air content;
2. a wet soil with less than 5% air content;
3. a workable soil defined as having more than 100 mbar moisture suction
4. a soil with workability for special operations defined as having more than 200 mbar moisture suction.

In order to avoid complications a very simple soil was used and a simple initial condition.

The soil

The soil used is a 'silt loam' with a straight moisture characteristic: $\Psi = 10\ \theta - 500$, where Ψ is expressed in mbar and θ in % by volume. Rijtema's (1965) expression for the $K(\Psi)$ relation was used:

$$K = K_o e^{\alpha\Psi} = 2e^{0.025\Psi}$$

The soil was taken to be uniform through the whole profile.

Initial condition

Regardless of the weather in the preceding summer, for every year the same initial situation on September 1 was used. This was a situation in which there is equilibrium with a constant downward flux of 1 $\mathrm{mm/day}^{-1}$. So the moisture content on September 1 was dependent on the drainage of the soil.

Weather

Rain and evaporation data were obtained from observations of the Royal Netherlands Meteorological Institute (KNMI) at De Bilt in the centre of The Netherlands. The first year was 1941/42, the last 1976/77. The year 1944/45 was omitted because some observations were lacking.

Evaporation data were calculated according to Penman and multiplied by 0.8. These data concerned periods of one month. The evaporation rates were distributed over the days proportional to radiation data.

There was no device to reduce evaporation data in dependence of moisture contents.

Drainage

Three drain depths and three drainage intensities were applied as shown in Table 1. Drainage intensity here is defined as drain outflow rate (cm/day^{-1}) divided by the height of the groundwater level above drain level (cm).

TABLE 1. Drainage intensities applied in the simulation with three drain depths

Drain depth (cm)	Drainage intensities (day^{-1})		
70	0.0100	0.0150	0.0250
100	0.0050	0.0100	0.0150
130	0.0050	0.0075	0.0100

Surface runoff occurred when there was more than 1.0 cm water upon the surface. The water in excess of 1 cm was removed at once.

Results

The analog simulation was carried out with soil layers of 10 cm, except in the drain depth of 130 cm, where some deep layers of 20 cm thickness were used.

The moisture content of the top 10 cm was continuously recorded with a line recorder. From these graphs the number of days that the moisture content did exceed a certain value was read.

Table 2 gives an example for the value of 45% moisture (5% air in the soil drained at 100 cm depth).

TABLE 2. Number of wet days (topsoil contains less than 5% air) in 35 years for drain depth 100 cm and three drainage intensities.

Month	Drainage intensity (day^{-1})			Mean rainfall minus evaporation (mm)
	0.005	0.010	0.015	
Sept.	103	71	57	22.4
Oct.	217	163	94	49.8
Nov.	368	182	125	62.8
Dec.	460	254	190	61.0
Jan.	405	179	127	64.8
Febr.	306	135	90	40.5
March	65	26	16	11.0
April	51	11	9	- 13.6
May	11	6	4	- 35.7
TOTAL	1986	1027	712	

The month December seems to be the wettest although in the months November and January there is somewhat more rainfall. The total amount of wet days comes to 57, 29 and 20 days per year for the three drainage intensities. So there is a pronounced effect of drainage intensity on the wetness of the topsoil, which was to be expected. In Fig.1 this effect is shown graphically. Drain depth seems to have an effect at least as important as drainage intensity.

The effect of drain depth and drainage intensity on the number of very wet days (topsoil with air content less than 2%) is shown in Fig.2. There it seems that drainage intensity is more important than drain depth. Nevertheless a very low number of very wet days will not easily be obtained with shallow drainage.

To reach a high number of workable days (θ < 40%, which means suction > 100 mbar) in March, drain depth is apparently more important than drainage intensity (see Fig.3).

This is even more pronounced in Fig.4 which considers the situation where the soil is dry enough for special operations, as seedbed preparation for potatoes and sugar beet. The most striking is the large difference between the drain depths 100 and 130 cm.

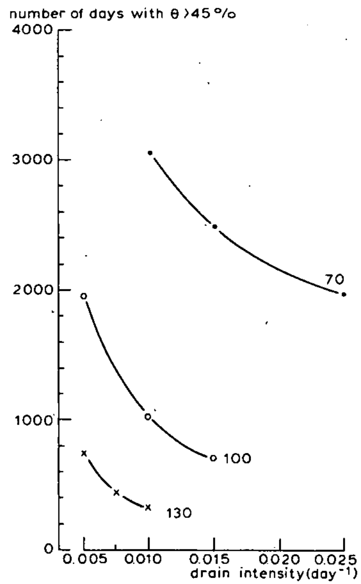

Fig. 1. Effect of drain depth and drainage intensity on number of days with wet topsoil; air content <5%, suction <50 mbar.

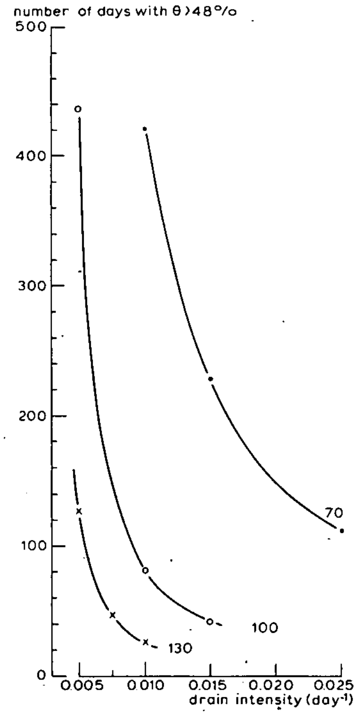

Fig. 2. Effect of drain depth (70, 100 and 130 cm below surface) and drainage intensity on the number of days with a very wet topsoil; air content <2%, suction <20 mbar.

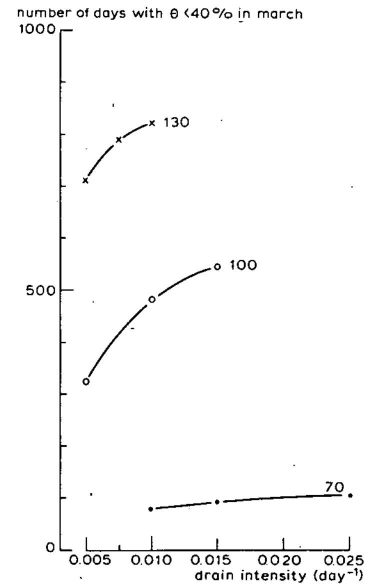

Fig. 3. Effect of drain depth and drainage intensity on the number of workable days; suction <100 mbar.

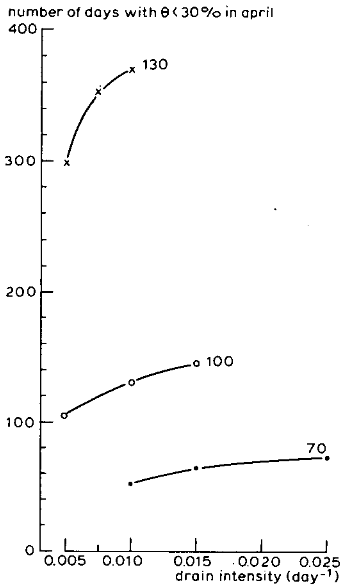

Fig.4. Effect of drain depth and drainage intensity on the number of days with workability for special operations, suction >200 mbar.

Discussion

The lines in Fig.2 are steep; the higher the figure-number the less steep the curves are. This means that the drier the reference value of moisture content is chosen, the less influence drainage intensity has. The conclusion of this somewhat schematized investigation is:

- To avoid very wet conditions drain spacing is an important factor, although the effect of drain depth is not negligible.
- To promote workability, the most important factor in drainage design is drain depth.

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Paper 1.12

DRAINAGE OF CLAY SOILS IN ENGLAND AND WALES

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Summary

There is an estimated drainage need on 2.6 million ha of agricultural land in England and Wales. Of the work currently being carried out 75% is on clayey soils.

In the clay soils of low conductivity, it is concluded that the only effective way of draining the subsoil is by secondary treatments of mole drainage or subsoiling over permanent pipe systems. These two operations are defined, and the conditions under which they are likely to be successful discussed.

In terms of current drainage design practices on these soils there is a large traditional factor, related to the particular part of the country. A more scientific approach is advocated, but due to the fact that most schemes are small (7 ha), there is an economic limit to the amount of pre-design survey that can be reasonably carried out. The options available are discussed, and a design philosophy suggested.

1. Introduction

There is a current estimated drainage need of 2.6 million ha of agricultural land in England and Wales. By far the greater proportion of work is on soils falling within the clayey textural classes, and its satisfactory drainage is therefore of paramount importance to British agriculture. The major factor influencing drainage design of these soils are:

i) hydraulic conductivity of the subsoil is generally very low and often decreases with depth

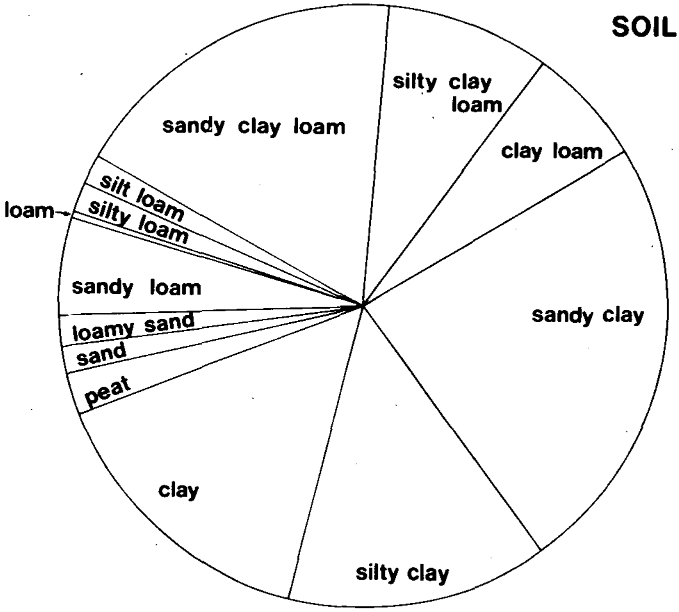
ii) because of i) some form of secondary drainage treatments are required in the form of mole drainage or subsoiling if the subsoil is to be effectively drained

iii) there can be considerable soil variation within relatively short distances

iv) average scheme size is only 7 ha, thus limiting the amount of time and effort that can be economically justified in individual site investigation.

2. Clay soils of England and Wales

Of all drainage work carried out in the UK 75 - 80% is on clayey soils, and as a soil group it creates the greatest problem in field drainage. The textural classes of all soils currently being drained are shown in Fig.1.



SOILS WHICH ARE BEING DRAINED

%	soil texture
15	clay
14	silty clay
24	sandy clay
6	clay loam
9	silty clay loam
17	sandy clay loam
2	silt loam
2	silty loam
0·26	loam
5	sandy loam
2	loamy sand
2	sand
2	peat

Fig.1. Soils which are being drained.

Due to the effects of glaciation the clay soils can vary over relatively short distances, and over considerable areas the conductivity tends to reduce with depth. Many of the subsoils have hydraulic conductivity (K) values less than 0.1 m/day and would require pipe spacings of 2-3 metres to effect adequate subsoil drainage. Table 1 lists some K-values of British clay soils, related to the Soil Series on drainage experimental sites. The Soil Series is the basic unit of soil classification used by the Soil Survey of England and Wales, and is defined as a class of soils with similar profile characteristics.

The conductivity values in Table 1 were determined by single auger hole method, and in all cases there is considerable range. The Drayton EHF experimental site, detailed in Appendix II, ranged from 0.006-0.02 m/day. If a permanent pipe system at 10 m spacing is taken as an arbitrary economic limit for drainage installation, then some form of secondary drainage treament (Section 3.3) is necessary on sites with K-values less than, say, 0.3 m/day, if subsoil drainage is to be achieved at realistic cost.

TABLE 1. Some hydraulic conductivity values of British soil (after Kellett, 1975), in order of K-value *

Site	Soil series	Texture	Depth (cm)	Median K-values
W Sedgemoor Somerset	Midelney	silty clay	(30-40)	0.00001
Samlesbury Lancs	Salop	clay	(70-87)	0.0035
Drayton EHF Warwicks	Evesham/ Denchworth	clay	(40-100)	0.0038
Cefn Coch (OS 270) Wales	Ynys	silty clay	(20-30)	0.01
W Sedgemoor Somerset	Sedgemoor	peat	(30-40)	0.02
Wilburton Cambs	Peacock	clay	(40-100)	0.03
Boxworth EHF Cmabs	Hanslope	clay/clay loam	(40-100)	0.03
Cross Moor Somerset	Allerton	silty clay	(40-100)	0.1
Cheddar Moor Somerset	Allerton	silty clay	(40-100)	0.23
Wrea Green Lancs	Clifton	clay loam	(50-65)	0.24
Bleadon Levels Somerset	Wentlloog	silty clay	(40-100)	0.28
Brooksby Leics	Hanslope - Ragdale	clay/sandy clay	(40-100)	0.32
Thornham Marsh Norfolk	Unclassified (Recent)	sandy loam/ clay loam	(45-100)	0.7

* *all values in metres per day*

3. Present drainage practices

3.1 Survey of current practices

Armstrong & Smith (1977) have examined current practices on 36,000 drainage schemes from data abstracted from the Ministry of Agriculture's records on schemes submitted for Government grant aid. The dominant drainage practices are shown on Fig.2 and the distribution of areas drained and average scheme size data on Fig.3 - the data is shown in terms of the Ministry's administrative divisions.

There is an area in East and Central England where the dominant practice is mole drainage over widely spaced drains. To the North and West of this area there is a transition zone of varying practice, with close drain spacing in the North and West of the country. The South East shows a departure from the East and Central England practices, but this probably reflects the soils which lie outside the zone of heavy glacial tills found in the other parts of the country. There is also a local practice of using straw over pipes in this region, and the analysis has shown this incorrectly as a *permeable trench backfill treatment*.

Critical examination of these data has identified inconsistencies of drainage design on similar soils in different parts of the country that cannot be explained by climatic or geographic variation.

Arising from this study national research priorities have been determined on the following basis:

High priority:	soils with gross differences in drainage practices	
Medium priority:	a)	soils with minor differences
	b)	soils uniformly treated, but with questionable effectiveness
Low priority:	soils uniformly treated, with demonstrable effectiveness.	

3.2 National study of soil water regimes

A national investigation to examine soil water regimes on drained and undrained sites, Rands (1976), Thomasson (1976), has shown that on many clay

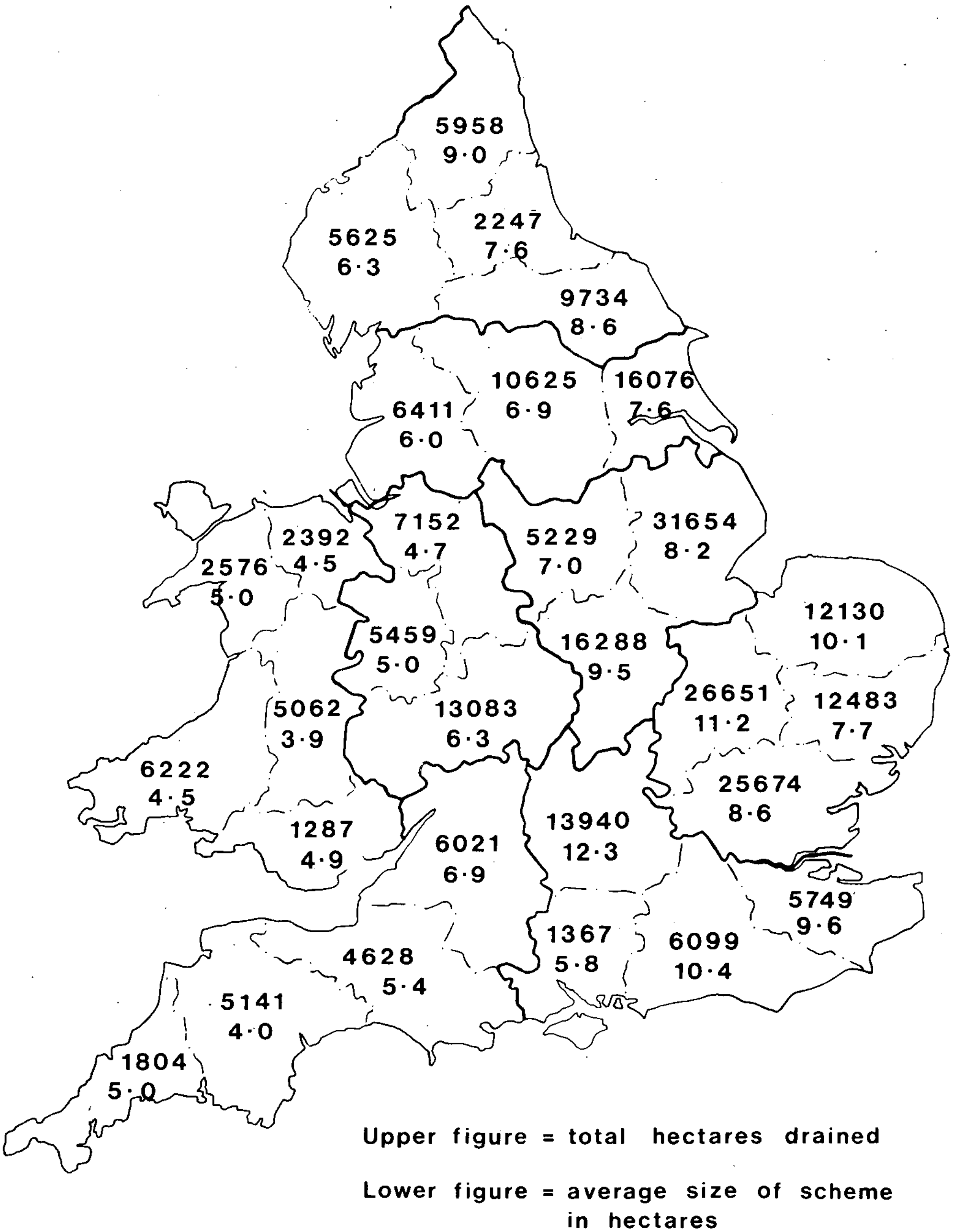


Fig. 2. Distribution of areas drained, England and Wales, during period 1.4.73 to 31.3.76.

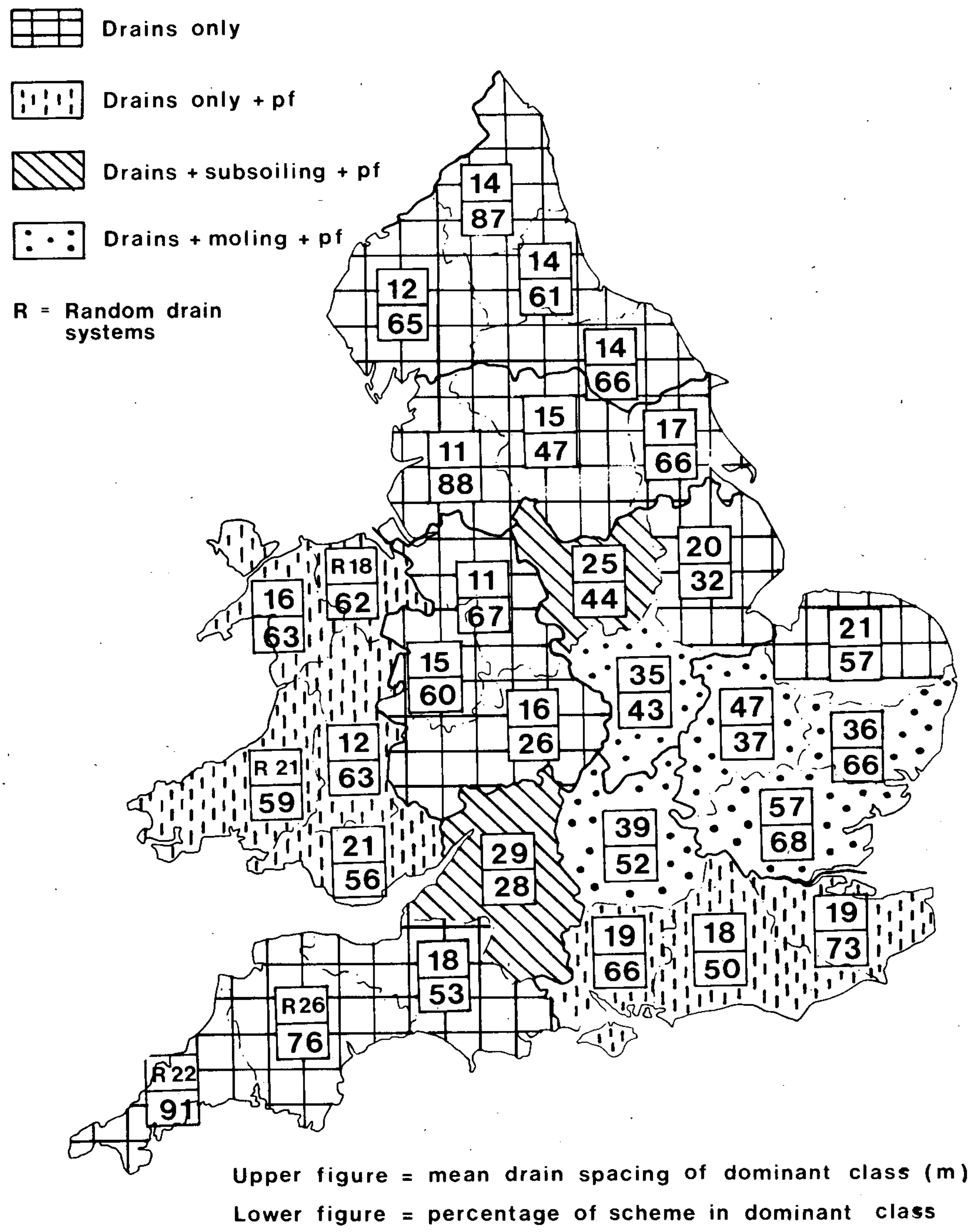


Fig. 3. Dominant drainage practices in England and Wales.

soils, close spaced drainage systems (10 m), in the northern and western parts of the country, are not draining the subsoil during the drainage winter and the effectiveness of some of these designs must be questioned.

There is a particular need to encourage successful subsoil drainage practices in some of the problem areas evaluated under the investigation discussed at Section 3.2. Many of these soils have K-values less than 0.3 m/day and requir either, even closer pipe spacing, or modification of the subsoil to increase the hydraulic conductivity. The soil water regime investigation is clearly showing that only surface drainage is being achieved on many soils and that there is scope for expanding secondary treatments of mole drainage or subsoiling into non-traditional areas.

3.3 Secondary drainage treatments

Moling

The technique of moling seeks to place inexpensive 'drains' at close spacing, intercepted by permanent pipe laterals at wider spacings determined by the stability of the soil to retain a channel. Present practice is to form a 75 mm diameter channel in the soil at a depth of 60 cm, see Fig.4. The timing of operations and stability potential of soils to retain a mole channel are critical factors and are priority research areas. Current advice has been set out in guidelines by Trafford and Massey (1975), and is summarized at Appendix I.

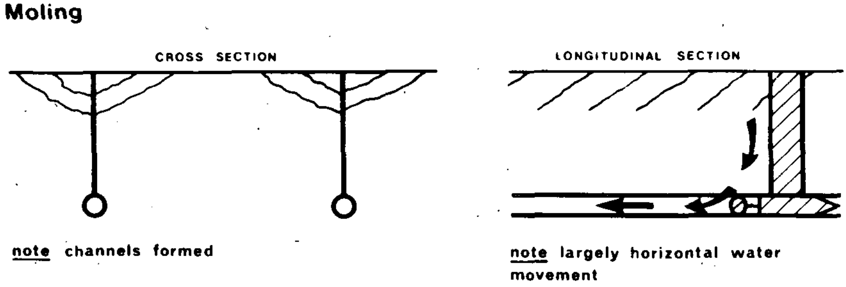
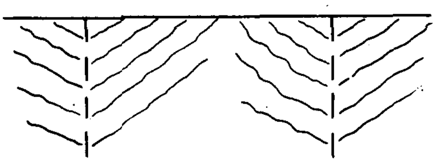


Fig.4. Diagrammatic illustration of moling.

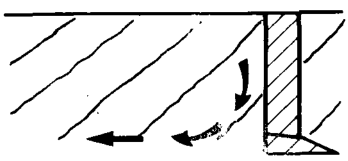
Subsoiling

Subsoiling in the drainage mode seeks to lift and shatter the soil peds to induce improved structure (Fig.5) and so improve the water movement to the permanent pipe system.

Subsoiling



note almost complete shattering of profile but no channel formed



note largely horizontal water movement

Fig.5. Diagrammatic illustration of subsoiling.

The basic difference between the mole plough and subsoiler is that the tine is wedge shaped tapering up to a square section; to achieve maximum shatter the subsoil must be dry.

Recent work by Spoor (1976) has shown that there is a critical depth, below which the failure is plastic with the subsoil being compressed rather than shattered (Fig.6).

The significance of this work is that effective subsoiling can only be achieved to a critical depth, which depends on soil moisture content, and rake angle and width of the tine - in UK conditions the critical depth is approximately 40 cm.

A recent examination of 50 subsoiling schemes revealed that in 80% a channel was created, indicating that the majority of operations were being carried out below critical depth and producing inferior mole channels. The effective shatter in the 20-40 cm zone was only 40% - a typical shatter diagram from adjacent tines is shown in Fig.7. This work was carried out with conventional single tine or multi-tine subsoilers, with mean implement foot width of 8 cm, and mean spacing between tines of 1-2 m.

Soil/tine interaction – critical depth

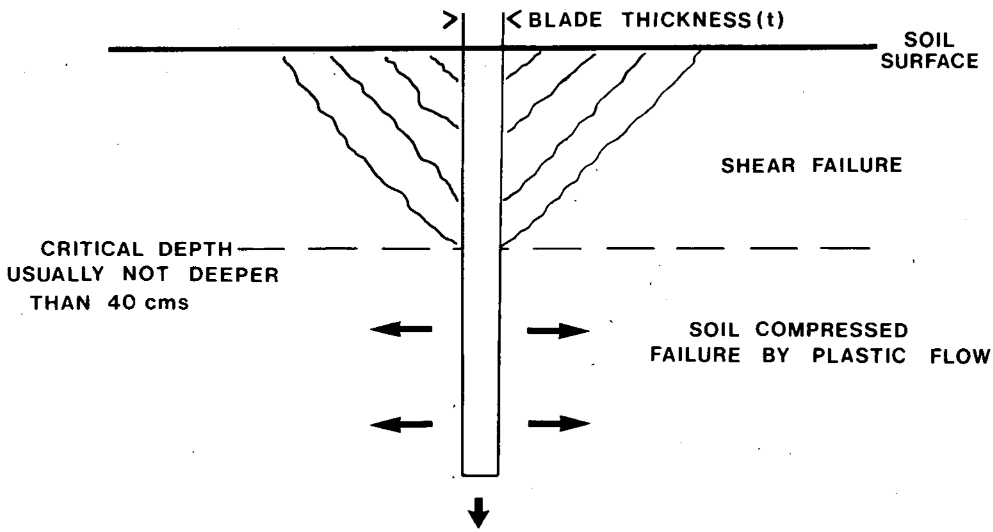


Fig.6. A diagrammatic representation showing upward soil failure in shear above the critical depth and plastic flow below this.

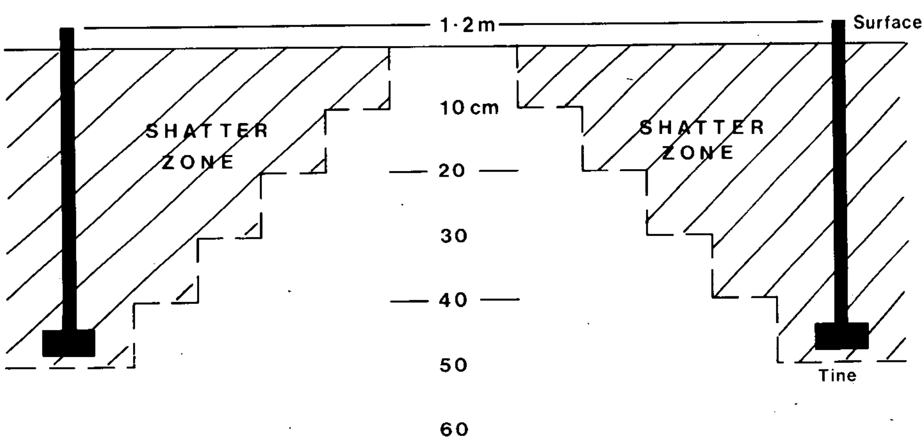


Fig.7. Typical shatter diagram for subsoiling giving the median shatter of 42% of the 20-40 cm zone.

Spoor's work shows that the addition of 'wings' to the subsoiler foot can significantly improve shatter with only small increases in draught requirements. The relationships of soil moisture conditions, implement design and depth of operations are high priority research areas.

3.4 Experimental evidence effectiveness of secondary drainage treatments

Over the last 7-8 years several field experiments on heavy land have been carried out to evaluate the hydrologic/economic factors of various drainage designs. May and Trafford (1977) have evaluated the results of the hydrological data at a site in the West Midland Region of England, the Drayton Experimental Husbandry Farm, Stratford-upon-Avon, Warwickshire. These results are representative of the other sites and the more important are presented at Appendix 2.

The Drayton experiment examined the relative performance of drains laid at 15 m, 30 m and 60 m spacing, with and without secondary drainage treatments of mole drainage and subsoiling. The best drained plots were those that had been mole drained over permanent pipe systems and showed an increase of 1 tonne/ha of winter wheat over the undrained control (Armstrong, 1976).

The main conclusions are:

a) The drainage treatment ranking in terms of hydrologic and yield performance is:

   moling + pipes
   subsoiling + pipes
   pipes only.

b) The permanent drain spacings were not in themselves significant. In the case of the mole drains these effectively acted as pipes at 2 m spacing and controlled the water table at 50 cm. This accords approximately with the theoretical spacing required with a K-value of 0.01 m/day at 50 cm depth.

c) Mole drainage channels may be expected to remain effective for 4-6 years provided that they are drawn under reasonable soil moisture conditions.

4. Practical design options

There is evidently a large 'traditional' factor in current designs practices on the clay soils. A good deal of research effort has been, and is currently being given to methods of draining the subsoil by secondary treatments. A prerequisite, however, is a more objective determination of the hydraulic conductivity values of the soil to enable further application to a satisfactory design.

4.1 Individual scheme investigation

The majority of design work is undertaken by Land Drainage Services advisers of the Ministry of Agriculture's Agricultural Development and Advisory Service, located at 30 divisional centres throughout England and Wales. Over recent years the Land Drainage Service has been receiving approximately 20,000 schemes/annum, with an average scheme size of less than 7 ha (see Fig.3).

Field experience has shown that due to the wide variation of soils, K-values by auger hole methods would require at least 5 wells/ha. Thus each scheme, average cost £2,000, would demand a very large time and resource commitment relative to the value of the work, and except in special situations could not be justified.

4.2 Design based on soil series

Based on the characteristics of the soil profile it is reasonable to suggest that for any soil series a fairly narrow range of field drainage design options could be stated. In the absence of hydrological measurements in an individual situation, soil series offers a reasonable design basis.

Using soil series as a basis Rands et al. (1973) suggested drainage designs for the soils in Eastern England. To overcome the lack of data on hydraulic conductivity, the soil series are allocated a 'class' based on the work of O'Neal (1952) - Table 2.

TABLE 2. Hydraulic conductivity classes

Class	Hydraulic conductivity value K in m/day
Very slow	less than 0.03
Slow	0.03-0.1
Moderately slow	0.1 -0.5
Moderate	0.5 -1.5
Moderately rapid	1.5 -3.0
Rapid	3.0 -6.0
Very rapid	more than 6.0

Application of this technique to a specific soil series is shown in Appendix III. This technique can be readily applied where soil maps are available, or where the soil series can be identified in the field in a non mapped area. The major limitation to use is that there can be a variation of soil within a defined series, and the application of a design layout from this may be too generalized in a specific field situation.

4.3 Design based on inferred 'K' values

Given that individual site determination of hydraulic conductivity is uneconomic, and that soil series approach has limitations in specific field situations, effort has been directed towards determining 'K' from site observations of more readily observable soil characteristics.

Work by Renger & Henseler on soils in Lower Saxony suggests that an estimate of hydraulic conductivity can be obtained from soil texture, structure, packing density and organic matter. Work is currently underway by Tring (1977) to examine the relationship of structure, packing density and hydraulic conductivity on a number of British soils.

Results are showing a correlation between packing density and structure (Tab. 3). This investigation is being given national research priority, and work is underway to measure 'K' values on the major soil series during the 1977/78 drainage winter. The work seeks to establish a relationship of a range of hydraulic conductivity values, with soil characteristics readily observable or obtainable by the drainage designer from a soil profile pit, and that does not require the presence of a water table.

TABLE 3. Comparison of soil structure/packing density values

Structure details	Arithmetic mean at 95% Confidence Level 1974-77
Medium prismatic moderate	2.01 ± 0.08
Medium prismatic close	1.96 ± 0.12
Coarse blocky close	1.99 ± 0.05
Coarse prismatic close	1.95 ± 0.06
Medium blocky close	1.94 ± 0.05
Medium prismatic free	1.91 ± 0.02
Fine blocky moderate	1.78 ± 0.07
Medium blocky free	1.68 ± 0.07
Single grain	1.30 ± 0.65
Medium blocky moderate	1.70 ± 0.07
Fine blocky free	1.61 ± 0.11
Coarse granular close	1.56 ± 0.03
Fine granular free	1.37 ± 0.04
Massive	1.33 ± 0.29

5. A design philosophy

Arising from the factors presented in the discussion paper a general philosophy is evolving for the drainage of the clayey textured soils of England and Wales. For drainage design purposes, clayey soils in England and Wales can be considered in 3 classes:

1. Soils with 'K' values in excess of 0.3 m/day, usually can be effectively drained by pipes at economic spacing.

2. Soils with 'K' values between 0.01 and 0.3 m/day, reasonably stable, and free of sand pockets and stones, may be drained with combined mole/pipe systems or where subsoiling above the critical depth can achieve an improved 'K' value then subsoiling/pipe systems.

3. Unstable soils and soils with 'K' values less than 0.01 will need to rely on surface drainage techniques.

Division of the drainage classes in this manner suggests that ranges of conductivity are more meaningful than specific values. The determination of these ranges is vital to improve the design of UK schemes and will be applied thus:

a) Design based on individual site measured hydraulic conductivity values. This method will only be used for exceptional cases.

b) Design based on Soil Series.
This method offers designers a useful guide at feasibility level, but there are limitations to the application due to field deviation from the standardised Soil Series profile.

It could be considerably enhanced if data on median 'K' values, together with the likely range, was readily available.

c) Design based on 'inferred' values of 'K'.
This approach would offer the UK designer the most realistic solution. It will enable the determination of meaningful design parameters at realistic costs.

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TRING, I.M. (Private communication.)

APPENDIX I

A guide to mole drainage
(after Trafford & Massey, 1975)

i) Stability to water

Previous experience, laboratory tests, or a crude test on a moulded lump immersed in water, are the best guides until advice on specific soils can be given. If the soil has a low hydraulic conductivity and is unstable to water subsoil drainage should not be attempted. If the soil has reasonable water stability underdrainage using moling may be considered.

ii) Timing of moling

The best practical advice would be that the soil at moling depth should be within the plastic range but not wetter than field capacity. This can be checked in two simple ways.

Plastic limit test

Take a small sample of soil from moling depth and attempt to roll out a thread $2\frac{1}{2}$ mm in diameter. If this can be done the soil is within the plastic range.

Field capacity test

Auger a hole to mole depth (55/60 cm) and leave for several days, making sure that surface water cannot enter. If water seeps into the hole then it is very likely that the soil is too wet to mole.

iii) Spacing of permanent drains

Permanent drains backfilled with gravel or similar material should be located in any natural depressions in the field. Closer spacings than this should be used if the soil is only moderately stable to water or there are sand pockets or other non-uniform areas. Maximum spacings might be 200 m and minimum spacings 10 m, although 20 - 60 m is more typical.

iv) Spacing between moles

A basic spacing of 2 - 3 m for the better clays (i.e. > 0.2 m/day) with spacings of 1 - 1½ m for the more difficult clays (i.e. < 0.2 m/day) would seem the most cost effective approach. The moles are the "drains" and closer moling will give a quicker drawdown and more effective control of the water table.

v) Depth of moles

Mole depths significantly shallower than 50 cm are not generally recommended as these are unlikely to give the desired soil moisture control. On some soils more stable clay may be found at greater depth and in this case it is desirable to take advantage of this up to about 70 cm. Beyond this depth the increased draft required may make the operation impracticable and it may be better to accept a shorter mole life.

vi) Frequency of remoling

No hard and fast rules can be offered as mole life seems to depend to a considerable extent on the conditions under which they were drawn and conditions soon afterwards. The important factor is to re-draw the moles before it is self-evidently necessary. As good moling conditions do not coincide with the land being free of crops in all years the precise life is not of practical interest. Most farmers who are successfully using moling re-mole a portion of the land (often 25%) each year that conditions are suitable. This means that re-moling is done every 4 - 5 years or so even though it might last 8 or 10 years.

vii) Mole gradients

It would seem to be good design practice to angle the moles with respect to the slope so as to have grades of 2 - 5% as a compromise between avoiding backfalls and erosion.

APPENDIX II

The Drayton experiment

i) Site details

Mapped as an Eversham Soil Series of the Lower Lias Clay (see Table 1). The site slopes south-southeast at 2%, and the cropping is arable with a mainly cereal rotation. Annual average rainfall is 625 mm.

ii) Layout

The experiment includes 11 plots, approximately 0.4 ha in size. There are 3 drainage treatments:

drains at 15 m - no permeable trench backfill
drains at 30 m - with and without permeable fill
drains at 60 m - with permeable fill

The pipes are laid at 80 cm depth.

There are 3 levels of secondary treatment:

none
subsoiling - 45 cm deep, 1.5 m spacing
moling - 55 cm deep, 2.0 m spacing

Two undrained plots are included.
The detailed layout is at Figure 8.

iii) Equipment

Raingauge, vane-in-orifice and tipping bucket flow meters, water table recorders, dipwells.

iv) Length of experiment

The experiment commenced in 1969 and after 7 years it is considered that the original objectives have now been met. Further experimental work is under way on the site looking at more closely the relative roles of moling and subsoiling.

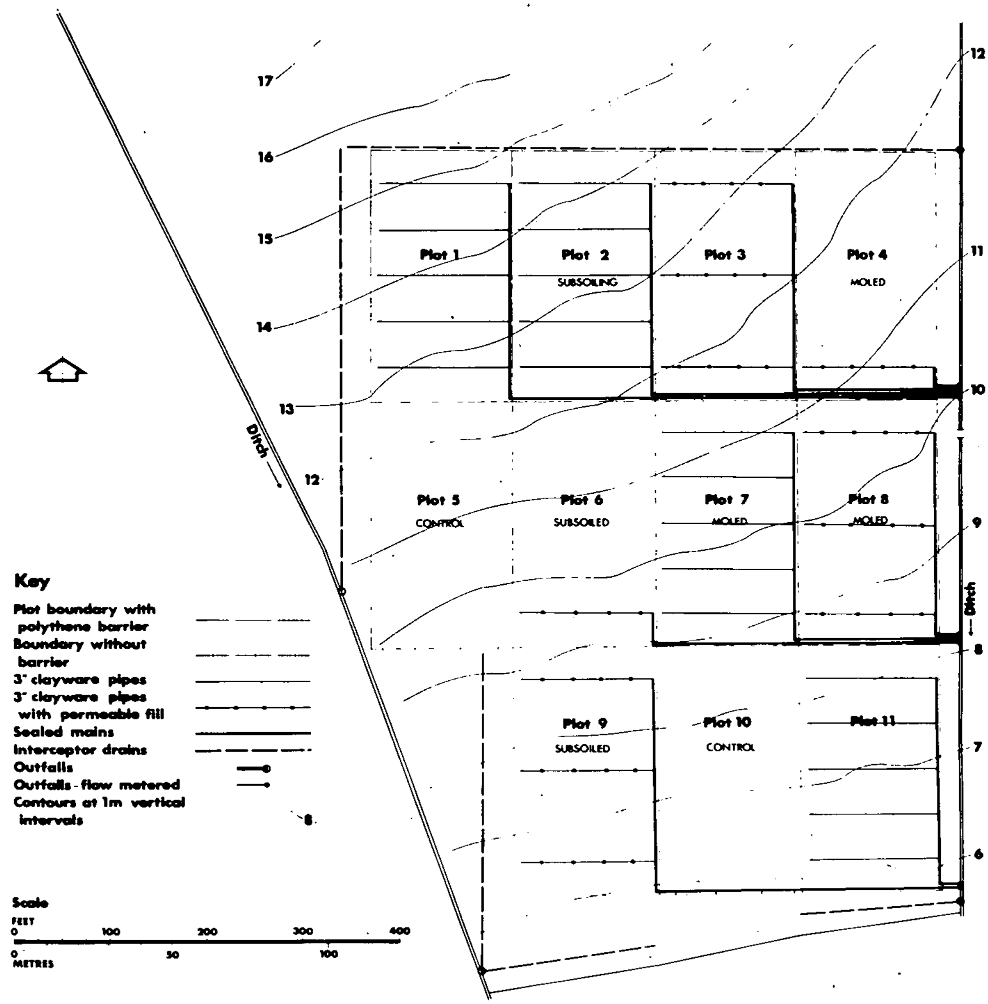

Fig. 8. The Drayton experiment. Site plan.

v) Hydrologic results

a) Drainage performance

With factors of slope, exposure, cultivations being equal, a measure of the relative efficiency of the various treatments is the ratio of actual drain flow (Qx) to potential drain flow (Qy). The potential drain flow is defined as that rainfall falling in the drainage 'winter' less any evaporation and any small moisture deficit.

Figure 9 shows the position when mean values $\frac{Qx}{Qy}$ are plotted against time for the 3 treatments of pipes only, subsoiling, and mole drains.

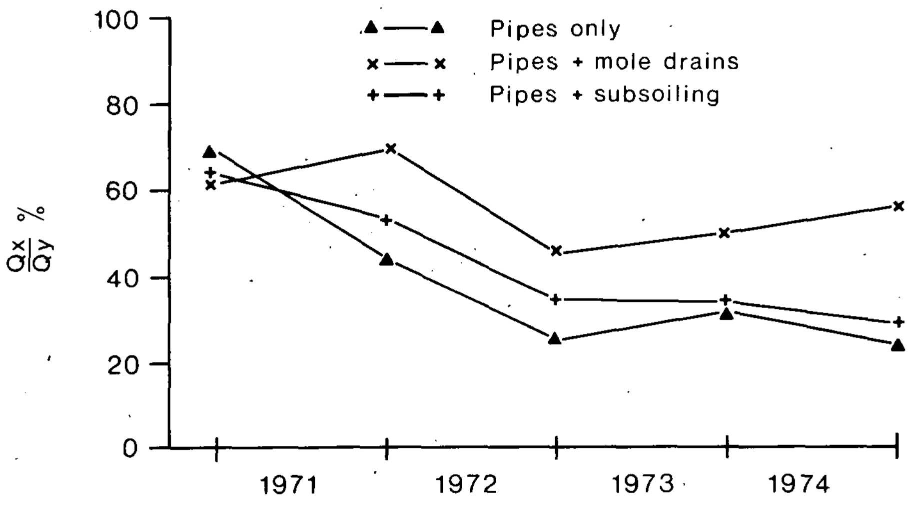

Fig.9. Mean drainage efficiency $\frac{Qx}{Qy}$ /time.

b) Water table control

The general pattern of water table control is that the moling and subsoiling treatments performed better than the pipes only. The results of 5 'winters' are shown in Figure 10.

c) Effect of pipe spacing

Pipes only - there was little variation in drainage efficiency $\frac{Qx}{Qy}$ and water table control despite a 2:1 variation in drain spacing. The explanation is likely to be that most water movement takes place in the topsoil with the drains acting only as interceptors.

ca) Mole drainage

With a 4:1 variation in drain spacing there was little evidence of reduced performance with the wider spacing, but there was a general failure of the mole channels after a 5-year life.

cb) Subsoiling

Although not totally conclusive there is evidence that the 15 m drain spacing performed better than the wider spacing. As discussed in Section 3.3 an 'inferior' channel was formed in the subsoil operation, and therefore although acting as mole channels, over the 30 m, 60 m drain spacing they are more likely to breakdown.

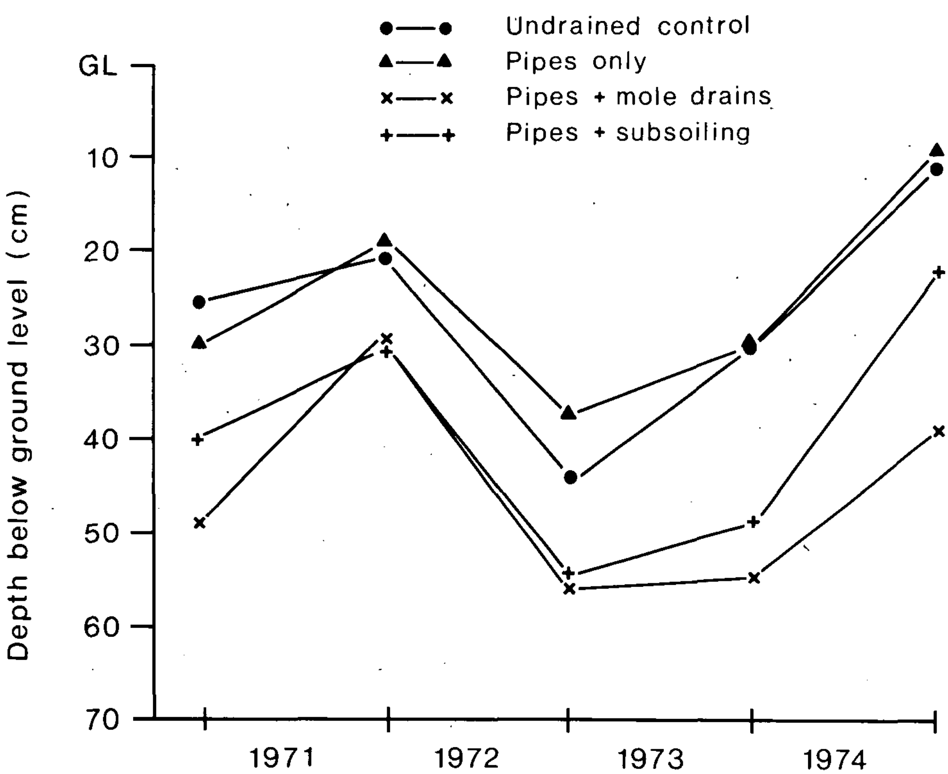

Fig. 10. Mean winter water table/time - all treatments.

vi) Crop yield results

Of the 6 years of record only 3 were for winter wheat - 1971, 1973 and 1974. Examination of the data showed that differences between years had an overriding effect on the absolute yield figures, and subsequent analyses were performed on data from which the annual means had been subtracted. The results show that no significant differences in yield can be attributed to drain spacings as such, but that secondary drainage treatment has a significant effect.

TABLE 4. Mean drainage effects. Increase in winter wheat yields over control (tonnes/ha)

Drain spacing	Secondary treatment			Mean
	mole	subsoiling	none	
15 m	1.10	1.07	0.58	0.91
30 m	0.90	0.16	0.78	0.62
60 m	0.98	0.48	–	0.73

APPENDIX III

Application of Soil Series to drainage design

Hanslope Series

(see also Table 1)

i) Soil Series

Hanslope

ii) Distribution of soil

Widespread in the Eastern Region.

iii) Topography

An upland soil found in rolling situations on plateau tops and on gentle upper slopes.

iv) Important drainage characteristics of the profile

A stable well structured calcareous clay permitting some natural water movement.

v) Soil/drainage design factors

Stability of topsoil	stable
Stability of subsoil	stable
Hydraulic conductivity class	slow/moderate
Variability	low
Occurrence of the soil in complex situations	infrequent

vi) Present drainage treatments

Slope is important and treatments tend to vary as follows:

	Plateau tops 1-2% slope	3% slopes and steeper
Drain spacing	40 metres	80 metres
Drain depth	80 cm	80 cm
Permeability aids	Moling	Moling
Permeable backfill	Normally used	Normally used

In some areas skeleton tile systems up to 200 m spacing have given satisfactory drainage. The excellent stability of the soil seems to make it an ideal moling soil.

vii) Recommended drainage treatment

As existing.

viii) Farming

Mainly cereals with some roots.

ix) Land capability class

Class 2.

Paper 1.13

RECLAMATION OF PEATS AND IMPERMEABLE SOILS

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Summary

The paper deals with the design and development of drainage techniques and equipment for the reclamation of peats and impermeable soils.

Blanket peats up to a depth of 0.5 m are usually ploughed to get a satisfactory peat/subsoil mix and an increase in surface strength. Drainage may also be required, depending on the permeability of the subsoil. Where the depth of peat lies between 0.5 m and 1 m, drainage is usually limited, by economic considerations, to surface grading to open collector drains. Where the peat depth exceeds 1 m, the "tunnel plough" or the "gravel tunnel" machine may be used to install an intensive drainage system at an economic cost.

Impermeable soils (K-values 0-0.1 m/day) require some form of subsoil disruption. Mole drains can be successfully installed on cohesive soils whose plasticity and shear strength characteristics are suitable. However there are no reliable criteria available for the selection of suitable mole drainage soils and investigations to establish an adequate classification system are urgently required. Soils unsuitable for mole drainage are subsoiled, ripped or have a system of "gravel-filled moles" installed. In the reclamation of all impermeable soils, particular attention is paid to surface grading to eliminate ponding and reduce depression storage to a minimum.

1. Introduction

A land drainage survey of Ireland (Galvin, 1969) showed that the major drainage problems in the country were Seepage and Springs (38%), Impervious Soils (33%) and High Water Tables (24%). Minor problems accounted for the remaining 5%.

The seepage problems usually occur in the free-draining regions and are generally solved by the application of conventional design principles. The biggest problem attaching to seepage investigations is that, due to our complex glaciations, many aquifers are discontinuous. As a result borehole pumping or the installation of a deep drain may occasionally result in the lowering of water pressure over a portion of the area rather than from the whole area. Water table problems are also solved by the application of established drainage principles and it is only where the hydraulic conductivity of the soil demands an uneconomic drainage intensity that problems arise.

Our reclamation efforts over the past number of years have therefore been concentrated on the investigation of impermeable peats and soils and on the design and development of drainage techniques and equipment for the economic reclamation of the materials.

2. Peatland drainage

General

Irish peatlands are generally divided into two major categories, blanket bogs and raised bogs (Barry, 1954). The blanket bogs cover extensive areas along the western seabord where the annual rainfall averages 1,400 mm, and also extend over high ground elsewhere, e.g. on the Wicklow mountains near the East coast. The total area of blanket bog is estimated by Hammond (1975) at about 870,000 hectares of which about 8,000 hectares are being developed for fuel production. The average depth of the deeper blanket bog is about 2.5 m and it is relatively uniform in composition throughout the profile. However, there are also extensive areas of shallow blanket bog varying in depth from 100 mm to 1 m. These areas are often adjacent to existing mineral farmland and are therefore a first priority for reclamation in a farm development programme.

The raised bogs are largely located in basin-type situations in the centre of the country where the annual rainfall is about 850 mm. The total area is estimated at 500,000 hectares. Of this, 100,000 hectares have already been cut over for fuel production and a further 60,000 hectares are in the process of fuel harvesting. In contrast to blanket bogs, raised bogs frequently exhibit in section a distinct sequence of peat types that reflects the changing environmental conditions under which they developed, from a low-moor stage, represented by reed-fen or woody-fen peat to raised bog. This develops above the influence of the groundwater and is characterised by the presence of peat derived from sphagnum, cotton grass and related species. During fuel harvesting, different peat types are exposed and the reclamation of the bog at any particular time depends on the degree of cutting that has taken place and on the underlying subsoil if the peat is less than a metre in depth.

Reclamation methods

The reclamation methods vary depending on peat type and local drainage conditions. Four very important considerations are:

a) depth of peat,
b) hydraulic conductivity of peat,
c) hydraulic conductivity of subsoil and
d) general surface slopes.

The latter is especially important where the hydraulic conductivity of the peat is low. Adequate surface drainage prevents large-scale surface ponding and reduces depression storage to a minimum. In this way the percentage run-off is maximised and the infiltration minimised thus resulting in the smallest possible addition to the water table for any given rainfall. This is especially so in the case of impermeable peats whose infiltration capacity is often exceeded for substantial periods by the rate of rainfall. Additional advantages, stemming from well-graded surfaces, are very much improved surfaces for sillage harvesting and a decrease in grassland poaching. Experimental trials on a peat cut-away showed very clearly that poaching always started at a minor hollow and spread from there (Galvin, 1972).

Blanket peats

The hydraulic conductivity of blanket peat varies from place to place but is generally of the order of 10 mm per day in the undrained condition. The reclamation methods used are conditioned by the depth of peat and the nature of the underlying subsoil. Where possible the subsoil is invariably mixed with the peat in order to strengthen the finished surface and improve its trafficability. Experience has shown that a mixture of equal depths of peat and subsoil gives satisfactory results.

Where the peat is up to 200 mm deep, basic reclamation involves ploughing to a depth of about 400 mm. This provides satisfactory mixing and sufficient loose material for subsequent surface grading. It also usually results in the disruption of any pan that may have formed under the peaty layer. However, if the pan occurs at a depth greater than 400 mm, provision must be made for its disruption either by subsoiling or by deeper ploughing. Where the subsoil is sufficiently permeable and where a high water table does not occur (this happens frequently on hill-land) no internal drainage is required apart from occasional water course to trap surface flow and intercept springs. However, where the subsoil is impermeable a system of internal drains is needed and is based on the hydraulic conductivity and other physical characteristics of the subsoil.

Details of the variety of drainage installations to cater for the reclamation of impermeable soils are discussed later.

For peats varying in depth from 200 mm to 500 mm, ploughing to a depth of 0.4 m to 1 m (deep ploughing) is needed to obtain satisfactory mixing, surface strengthening and sub-surface disruption. Deep ploughing is a relatively expensive operation and since the overall expenditure on reclamation is limited to approx. £850 per hectare, the combination of deep ploughing and piped drains can only be accommodated where the required drain spacing is greater than 40 m. Where the subsoil is permeable and a high water table does not occur, deep ploughing used in conjunction with intensive surface grading (aimed at eliminating all local depressions and hollows as far as possible) is normally sufficient. Where the subsoil is impermeable or where a high water table occurs, the peat/subsoil mixture must be stable and its permeability such that it can be drained at a spacing of 40 m or greater. If this does not obtain the scheme is considered uneconomic.

Where the peat depth exceeds 0.5 m, subsoil mixing is excluded on the basis of economics. For depths ranging from 0.5 m to approx. 1 m, the provision of a satisfactory internal drainage system is generally economically infeasible. However peats of this depth often occur on sloping land and in the absence of a high water table, satisfactory reclamation can be attained by careful surface grading, combined with occasional surface water catchment drains. This system can also be successfully applied to deeper peats under the same conditions. For blanket peats deeper than 1 m and affected by a high water table, a drain spacing of between 1 m and 4 m is required due to the basic impermeability of the peat. The drainage system involves the installation of main collector drains strategically placed in natural hollows along the contours. The provision of the intensive system of internal drains (at the 1 m to 4 m spacing) is another problem. Conventional piped drains could be used but the costs involved would be completely unrealistic especially in view of the fact that blanket peat even when intensively drained is by no means an ideally trafficable medium. Ordinary mole drains are unsuccessful as they collapse within a very short period. In the circumstances two methods have been developed aimed at providing reasonably good water table control at an acceptable expenditure.

One of these methods (Armstrong, Burke & Quinn, 1960) involves the excavation of a tunnel section (380 mm deep × 280 mm wide) at a depth of approx. 800 mm. The excavated peat is extruded from the top of the machine and can be deposited on the bog surface as a ribbon of peat or macerated by activating a PTO-driven macerator on top of the tunnel plough. This machine was originally developed in 1959 and used with varying degrees of success until the early sixties. It has since been re-designed by Burke and Grubb (1978) and is now a far more effective machine. There is one major difficulty involved in the use of this plough. It works on the basis of extruding a band of peat and therefore tends to fail when the plough encounters a section of bogland that is either too slurried to allow for proper extrusion or too fibrous in relation to its shear strength. However, a system for measuring the vane strength of the peat prior to tunnelling using a modified field vane has been devised. This method has been developed by measuring the vane strength of peats that have been successfully and unsuccessfully tunnelled and from these figures an empirical method for determining the suitability of a bog for tunnel drainage has been devised by Burke & Grubb (1978). In the recent past, very profilic rooting systems have been obtained by using the tunnel plough for forestry drainage. The big advantage has been the aeration provided at the 400 mm - 800 mm depth and also the space provided in and around the tunnels for root development and proliferation. The Forestry Departments in Ireland, and Northern Ireland have been very impressed by the results and are considering using the tunnel plough drainage system as far as possible in future blanket peat plantations.

Because of difficulties associated with the tunnel plough in the early sixties a more positive drainage system was developed for blanket peat in 1963. The method involved the installation of a band of gravel (approx. 100 mm × 80 mm) on a layer of polyethylene at a depth of approx. 800 mm. A prototype machine to install this drain was developed by Burke and McCormack in 1969. After further trials and experience this machine has been redesigned by Burke and Grubb and will be used on commercial drainage installations in 1978. This method involves the installation of approx. 25 cubic metres per hectare of 16 mm clean single-sized gravel chippings as a series of gravel bands at a spacing of 3.5 m. This is the minimum spacing justified by existing economic considerations and gives acceptable water table control on grassland during the summer months. It is always combined with as much surface grading as feasible within the monetary constraints of each job as the elimination of depression storage is particularly important on schemes where the water table was close to the surface before drainage.

Raised bogs

The reclamation of raised bogs follows the same general principles outlined for the blanket peats. However, as many of these peats have been cut away to varying degrees, the permeability and strength of the peats exposed at the different depths varies as also does the depth of the water table below the surface. The sphagnum peats are soft and have strength characteristics similar to those found in blanket peat, and must be treated accordingly. The fen peats, however, are stronger and can provide an ideally trafficable medium, without mixing with the underlying subsoil. They are also more permeable and as the wood content increases the hydraulic conductivity can increase to 1 or 2 m per day. These peats are obviously very valuable and are used for arable cropping with minimum drainage. A disadvantage is that under an arable system, surface shrinkage usually amounts to 50 mm yearly so that a depth of 2 m to 3 m is required for continuing arable production.

3. Drainage of impermeable soils

The impermeable soils referred to in the context of this discussion are those with K-values ranging from 100 mm per day to almost zero. Irish soils of this type are derived from boulder clays formed from a variety of glaciated parent materials. Because of this, they are often very variable in composition and the fine (clay size) particles contain an appreciable percentage of finely ground non-expandable minerals (Mulqueen & Burke, 1967). Many of these soils have been overconsolidated under the heavy overburden of ice and in some cases the bulk density can reach 1,620 to 1,650 Kg/m^3. Some of the soils occur under raised or blanket bogs but where the depth of overlying peat is less than 0.5 m the efficiency of the drainage method used is directly related to the efficiency of subsoil drainage.

Under Irish climatic conditions these impermeable soils are not suited to tillage operations. Apart from their sticky nature they are usually stoney and completely unsuited to continuous cropping. In these circumstances they are invariably used for grassland and as such there is a limit on the reclamation expenditure that can be justified by the expected returns from efficient grassland farming. As of now the optimum returns result from intensive dairying and as previously stated the maximum justifiable expenditure is approximately £850 per hectare. This figure includes cultivation, reseeding and any necessary fencing, etc. required to bring the land into full production. The spacing of conventional drains is therefore limited to a minimum of approximately 30 m by these economic factors and the required drainage intensity for soils with K-values less than 100 mm per day cannot be accommodated by conventional drainage systems.

In a situation as outlined, where conventional drainage methods cannot provide an economical system, the approach must be to alter the subsoil (improve its hydraulic conductivity and reduce its water holding capacity) so that a satisfactory system can be provided with more widely-spaced piped drains. The traditional methods of achieving this have been the installation of mole drains or by subsoiling. The former enables one to provide a very intensive drainage system by substituting moles for the conventional pipes but this requires a soil in which the moles remain stable over a sufficiently long period. Subsoiling works on the principle of disrupting the subsoil to such an extent that its K-value is increased sufficiently. As with mole drainage a prime necessity for satisfactory subsoiling is a soil that, once it has been shattered, will maintain that shattered state over a sufficiently long period. No detailed economic investigation has been undertaken on this aspect but our experience leads us to the conclusion that mole drainage or subsoiling, to be viable, should not need to be repeated at intervals shorter than 5-6 years

Soils that are eminently suited to subsoiling are those of a cohesionless nature, and cohesive soils of low plasticity and high sheer strength. When shattered, these soils tend to remain in that conditions and do not puddle or slurry under the newly established drainage regime. Cohesive soils may be suitable for traditional mole drainage if their plasticity and shear strength characteristics are such that a well-formed mole can be established which will retain its shape for at least 5 years. Researchers in many countries have carried out investigations to try and establish exact criteria for good mole drainage soils but in our experience no satisfactory criteria have as yet been established. We have carried out detailed investigations on a number of soils over the past two years with the same objective. These investigations are not complete but to date we have found no simple test for establishing the suitability of soils for mole drainage. In Fig. 1 the textural analysis of soils that invariably mole well is shown on curve A. Curve B indicates the textural composition of a number of soils which are sometimes successfully moled but which on other occasions break down badly within one year. It would appear that these "borderline" soils can be successfully moled when all the conditions (soil moisture content, subsequent weather conditions, subsequent land use, etc.) are optimum. However, in the absence of optimum conditions complete failure is commonplace. There is no suggestion that textural analysis alone can be used as a yardstick for establishing the suitability of soils for mole drainage. The curves are cited mainly to emphasise that a large number of soils whose textural analyses are almost identical with curve B (and whose plasticity characteristics are also practically identical) are on the borderline of success or failure. From a practical viewpoint these soils must be considered unsuitable for mole drainage and a far more expensive alternative drainage system installed. Further investigations into all aspects of soil suitability for mole drainage are urgently needed.

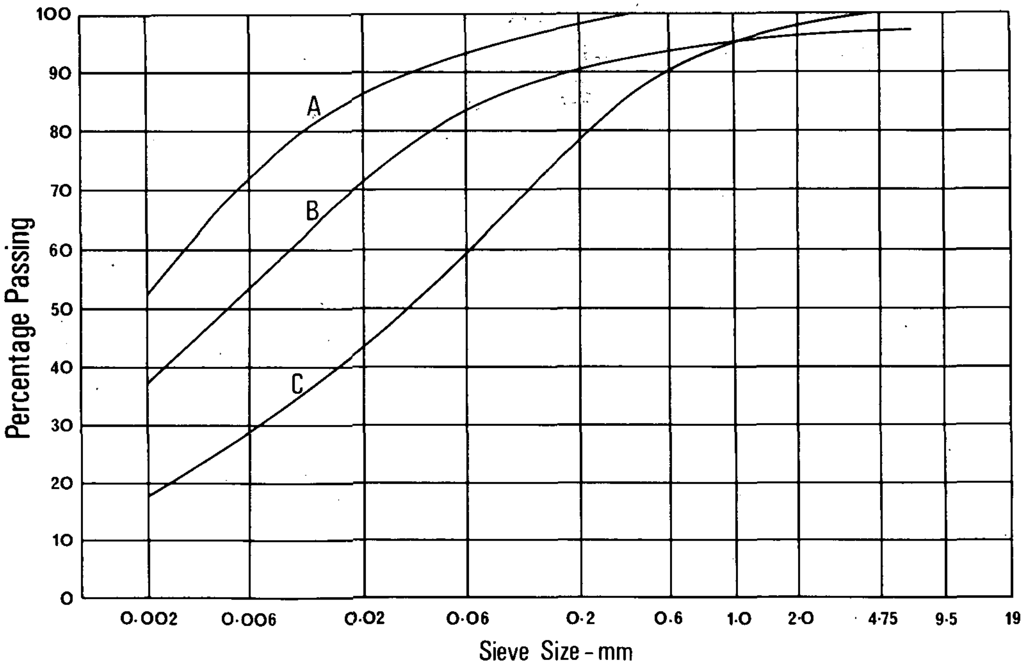

Fig.1. Textural analysis of 3 soils.

Soils which are not suitable for conventional mole drainage and are of such high plasticity that they cannot be adequately shattered by subsoiling have been successfully reclaimed over the last four years using a system of gravel-filled moles (gravel moles) designed by Mulqueen and Prunty (Mulqueen & Harrington, 1976). In this system, the conventional moles are filled with 16 mm single-sized clean washed gravel. They are installed at 1 m centres and at a depth of 0.5 m. Catchment drains are installed at a spacing of 60-100 m, depending on field size and slope. This system has worked extremely well on a number of very difficult soils.

A third curve (C) on Fig.1 shows the textural analysis of soils that are too coarse-textured for mole drainage and because of their extremely high bulk densities (1,620 to 1,650 Kg/m^3) and stoney nature are also unsuitable for ordinary subsoilers. These soils have been successfully reclaimed, by using heavy-duty road rippers mounted on 180 H.P. Dozers to disrupt the subsoil to a depth of 600 mm (Galvin & McGrath, 1977). The tool bar is usually equipped with 3 tines at 1 m centres and the ripping is carried out so as to shatter the subsoil at 0.5 m spacing. The ripping is done across the contours and catchment drains are installed along the contours at spacings varying with the type of subsoil and the steepness of the slope but generally of the order of 30 to 40 m. As with all reclamation work on impermeable soils particular attention is paid to surface finish and uniformly graded even surface is provided as far as possible.

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Paper 1.14

REGULATION OF WATER REGIME OF HEAVY SOILS BY DRAINAGE, SUBSOILING AND LIMING AND WATER MOVEMENT IN THIS SOIL

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Summary

The amount of runoff from two drain fields on pseudogleys of loess and clay with increasing drainage distances combined with vertical soil drainage and loosening of the subsoil (0.95 m depth, 0.75 m vertical distance) were compared with the amount of runoff from a conventional drainage without subsoiling. The highest yield was obtained at a drainage distance of 80 m. This mode of drainage is not only good for farming but also for environment.

By using different materials for filling the drain ditch, we found, that not gravel but excavation and topsoil gives the greatest runoff. The water movement in soil to a drain was observed by measuring the natural electric streaming potential. We found, that water movement follows not the usual expected flow.

1. Introduction

A site with a plant-physiological acid reaction needs liming of about 50 dt/ha. If a soil is compressed too, we use liming and subsoiling for melioration. Finally, if a site is wet, we need a drainage. After Schmid (1973) and Rager & Schmid (1970), we call this kind of melioration "melioration with three grades".

Regulation of water regime is as old as modern farming. Primarily only the amount of runoff is important. Today retention of soil moisture for dry periods especially by pseudogley is important too. Economic aspects must be considered.

2. Drainage experiments

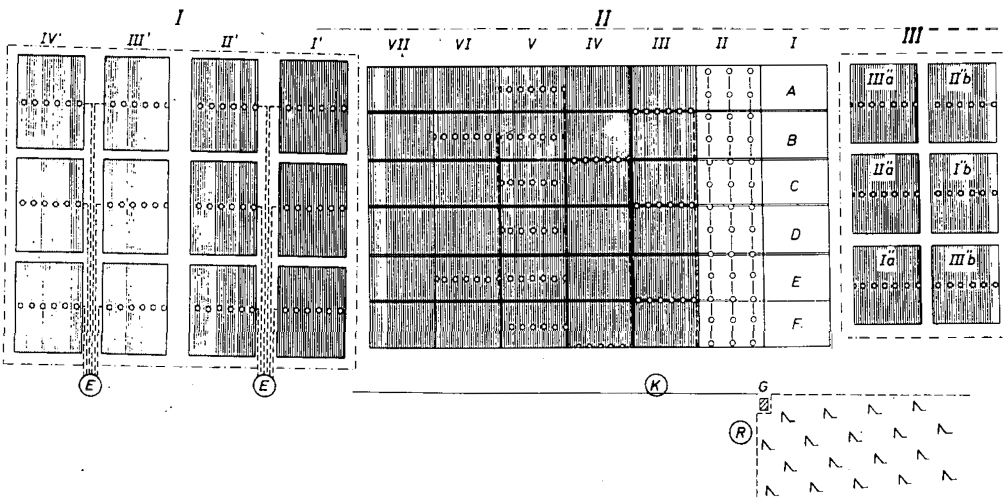
In 1967 our Institute began to install two drainage experiments on different, heavy soils.

Table 1 shows some data of the soil near Ottenhofen.

TABLE 1. Fraction of particle size of the soil near Ottenhofen

	sand	silt	clay
0 - 24 cm	8.2	78.2	13.6
24 - 36 cm	10.3	60.4	29.3
36 - 100 cm	4.4	72.1	23.5

The plan of one of the drainage fields near Ottenhofen in the east of Munich you see in Fig.1. Please have a look at part II and I in the centre and the left side of Fig.1. In part II you see seven sections (I to VII). In the white section (I) with the letters A, B, C, D, E and F there is no drainage or loosening but liming at A, C, and E. In the following section to the left we have a conventional drainage (10 m) with clay pipes at a depth of 1 m. The runoff from this section is collected in the south and registered in a cellar R. Near one of these drains the water movement to the drain was studied by observing the electric streaming potential (Schuch, 1963, 1966 and 1969, Schuch and Wanke, 1967, 1967a, 1968, 1968a, 1969 and 1969a) as described under 5. In the following sections (III, IV, V and VI) the distance of the drain ditch with clay pipes is varied from 40 m (III), 80 m (IV), 20 m (V), and finally 60 m (VI). Gravels were used for filling the ditches in section III, IV, V and VI. In the section VII on the left hand side there are no ditches with pipes. The hatching shows the direction of subsoiling at a depth of 0.8 to 0.9 m and a vertical distance of 0.75 m. The runoff from every section III, IV, V and VI is registered in the cellar R. To prevent water inflow from catchment, a cut-off drainage is located around the experimental field (shown in Fig.1). I mentioned in section VIII there is no drainage. The water level is high and we have an influence to section VI. Before we discuss the amount of runoff I have to say that all parts A, C and D are limed. We propose 40 - 80 dt/ha burnt lime with a fraction of < 2 mm. In the experiment near Ottenhofen we used at the beginning more, but it was too much and so we have had a depression of crop in the first years especially potatoes. We use a crop rotation between the parts A + B, C + D, and E + F. Average precipitation over a long time amounts to about 831 mm/a.



I. Field testing different material for seepage water in the drain ditches; o o o clay pipe, - - - plastic pipe, E = excavation, hatching shows the direction of subsoiling.

II. Field testing different drain distances (2.63 ha) A, C and E are limed.

III. I" a+b is subsoiled with a motioned tool; II" a+b is subsoiled with a fixed tool; III" a+b is subsoiled with a motioned tool combined with deep manuring. K = climate station, R = measuring cellar, G = accounting room.

Fig.1. Drainage field near Ottenhofen in the east of Munich.

3. Runoff

Precipitation N and amount of runoff A from the drain field near Ottenhofen during 1970 to 1977 is shown in Fig.2. The white column demonstrates the amount of runoff during the single month, the black column shows the greatest amount of runoff during one day. The number beside A is the amount of runoff

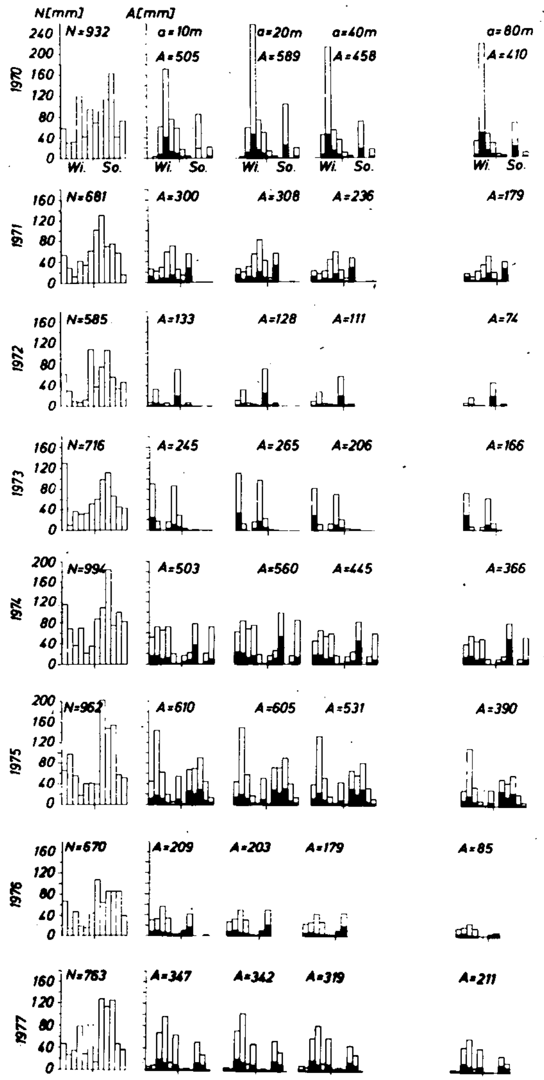


Fig. 2. Precipitation and runoff during the years and the months 1970-77 and the greatest daily runoff (black column) from a drainage field near Ottenhofen.

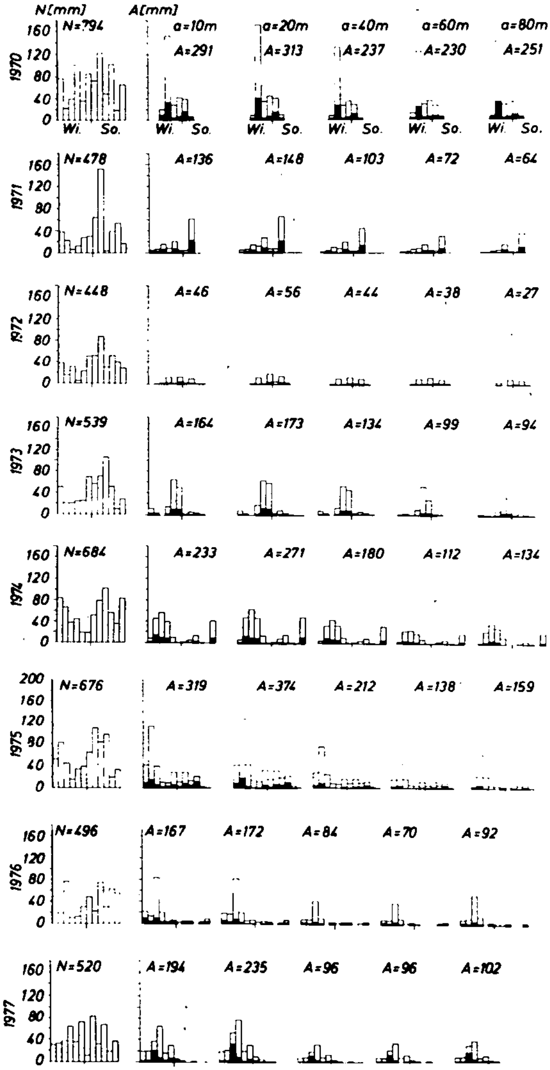


Fig. 3. Precipitation and runoff during the years and the months 1970–1977 and the greatest daily runoff (black column) from a drainage field near Ellingen.

during one year (mm/a). Please notice while comparing the single values, that the drain field with a distance of the ditches of a = 10 m is not subsoiled, but the fields with a = 20, 40 and 80 m are subsoiled. You see all the time that the field with a = 20 m has the greatest runoff during a year. The field with the next greatest runoff is that with a = 40 m. The least runoff during a year shows the field with a = 80 m. At the field near Ottenhofen all years long we get the same order. The runoff from the field with a = 10 m (without subsoiling) sometimes gives a little greater runoff than the field with a = 20 m (with subsoiling) sometimes a little smaller. But the most important result you see in the black column. If there is enough precipitation or rainfall, all drain fields show a very great and nearly the same amount of runoff (black column) even 8 years after subsoiling. We can say, the amount of runoff from the field with great drain distance and subsoiling is by enough rainfall as great as from a field with a drain distance of 20 m. But the water absorption capacity of the fields with greater distance is during a year greater than those with a small drain distance with or without subsoiling. These results are representative for soils like the soil of our experimental field at Ottenhofen.

And now the results of our drain field near Ellingen, about 100 km in the north of Munich.

Here we have another, much more heavier, soil.

TABLE 2. Fraction of particle size of the soil near Ellingen

	sand	silt	clay
0 - 20 cm	19.2	55.7	25.1
20 - 32 cm	15.7	44.7	39.6
32 - 65 cm	10.6	26.3	63.1
65 - 100 cm	4.7	36.2	59.1

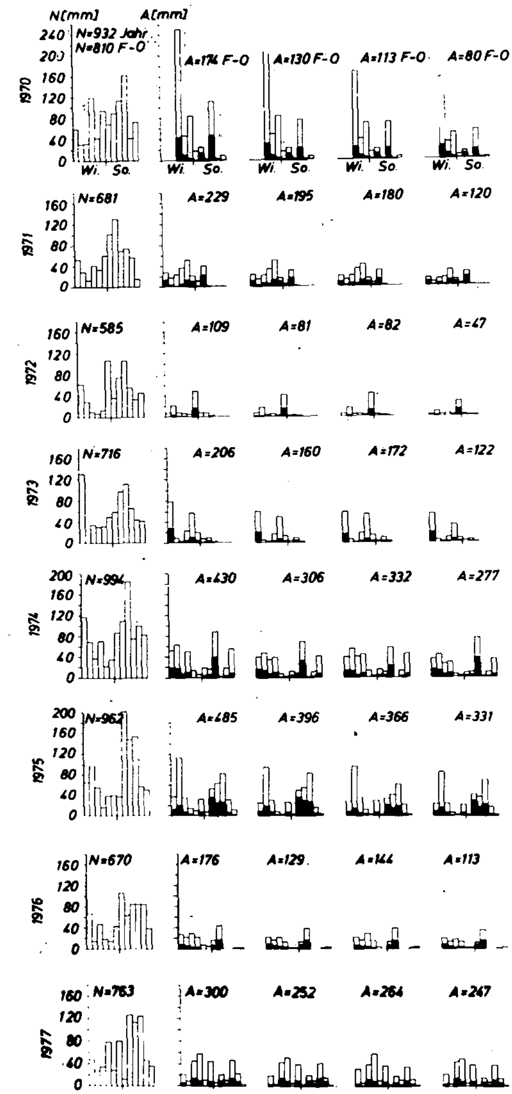


Fig. 4. Precipitation N and runoff A from 4 drainage fields near Ottenhofen. The ditches of the fields were filled with excavation, topsoil, styromullsphere and gravel (from the left to the right side).

F-0 = February - October; Jahr = year; Wi = winter half year; So = summer half year.

The experiments are nearly the same as described in the plan of Fig.1 (Schuch 1970; Schuch, Jordan 1971, 1972, 1973 and 1975). We only took an arrangement without any influence of water inflow of catchment. The results of runoff are given in Fig.3. You see that the average precipitation is smaller than near Ottenhofen. Average precipitation over a long period amounts to about 659 mm/a. It seems that runoff, especially the greatest daily runoff gets smaller and smaller in the course of time. Even in the wet year 1975 a plough for deep cultivation (40 cm) was necessary to improve water conductivity especially in the upper layer.

4. Backfill

Because it is necessary to make cultivation as economic as possible we look for another cheaper material for filling the drain ditches. We used four materials to fill the ditches; excavation, topsoil, styromullsphere (an artificial material), and gravel 7 - 15 mm.

The amount of runoff during the years 1970 to 1977 and of each month you see in Fig.4. We were rather astonished that runoff from the fields with ditches filled with gravel and styromull were smaller than runoff from the other fields. Fig.4 shows this tendency all the time. At first we thought that there was a mistake, but there does not seem to be.

We found this result proved at our parallel experiment near Ellingen. The diagram in Fig.5 shows the results. Drain ditches filled with gravel show till now less runoff than the ditches filled with excavation or topsoil. If this result remains in future, we only use the much more economic excavation for filling the drain ditches. I must remark that excavation was dried and crushed before filling the ditches.

5. Soil moisture movement

Besides the practical aspects under point 2 and 3 we studied the water movement to a drain by observing the natural electric streaming potential. Each water movement in soil causes an electric streaming potential (Schuch, 1962). This phenomenon was used to determine the direction of water movement in different kinds of soil by Schuch & Wanke (1967, 1967a, 1968, 1968a, 1969, 1969a)

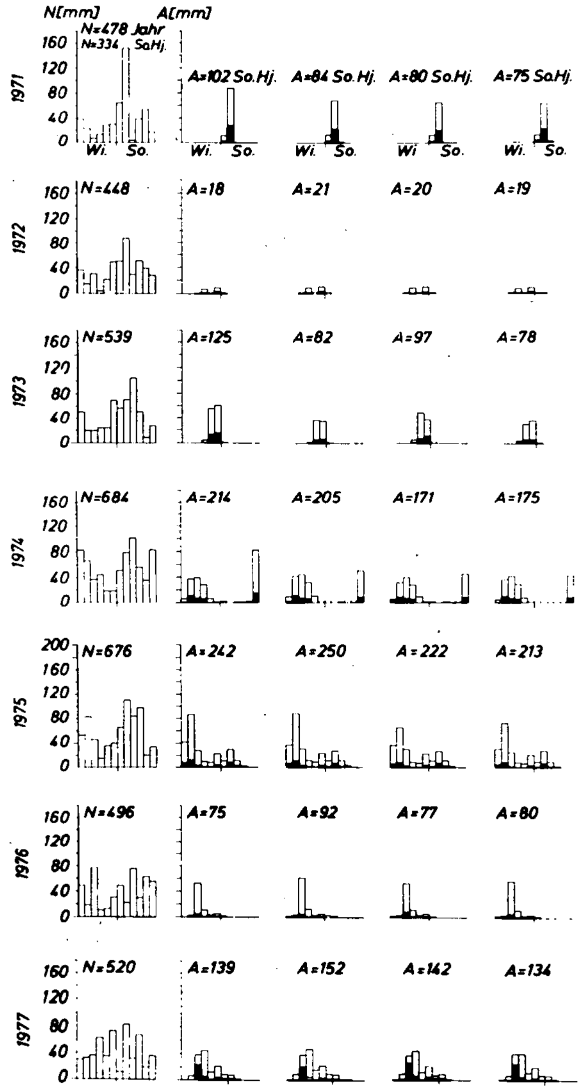


Fig. 5. Precipitation N and runoff A from 4 drainage fields near Ellingen. The ditches of the fields were filled with excavation, topsoil, styromullsphere and gravel (from the left to the right side).

Jahr = year; SoHj = summer half year.

To get a glance at soil water or soil moisture movement in the drain field near Ottenhofen a configuration of measure lines shown in Fig.6 was chosen. On the right side you see the drain 80 cm deep under the surface. On the left you see the position of 24 electrodes. 12 electrodes give a three-component-scheme. The others are situated at a greater distance and a larger depth.

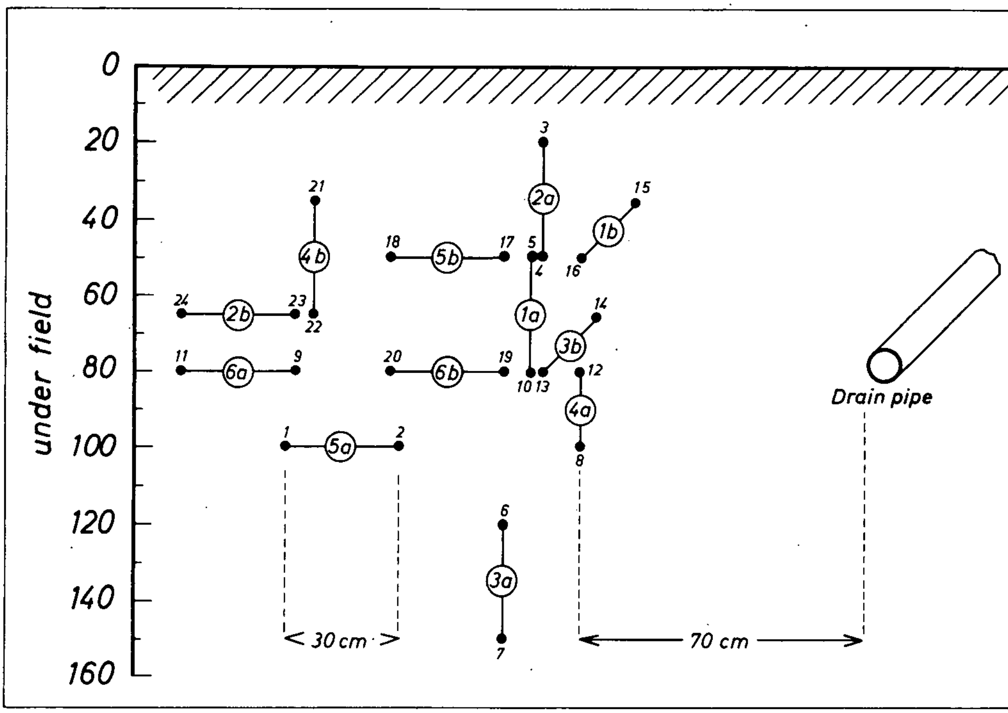
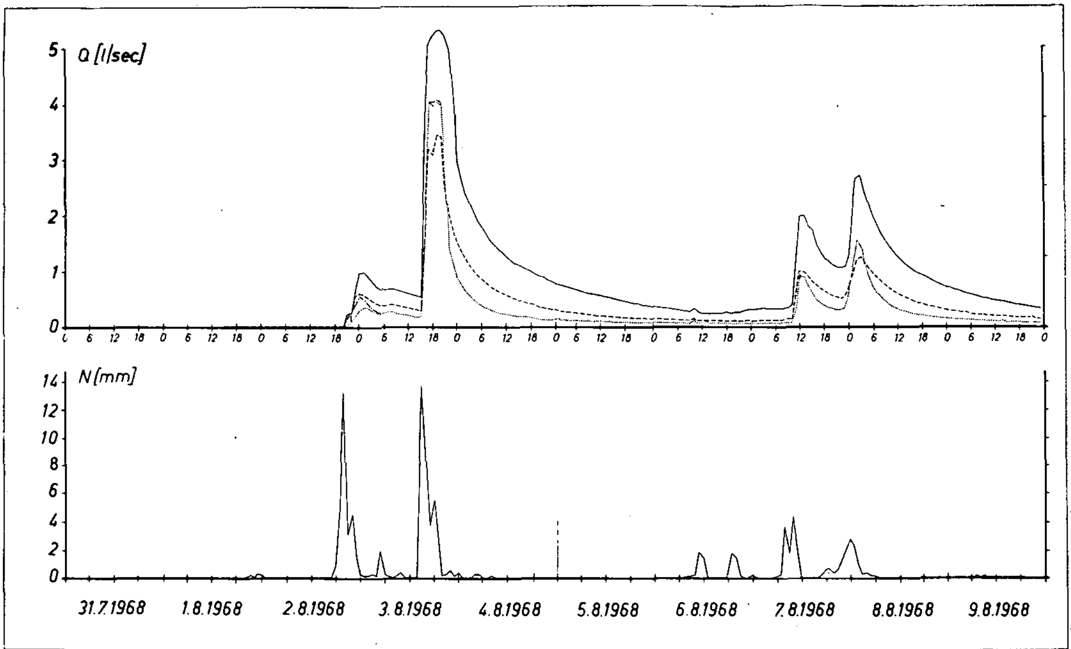


Fig. 6. Position of electrodes near a drain.

The amount of rainfall N and runoff Q of the drainage can be seen in Fig.7 (Schuch, 1970).



——— Q by a drain distance of 20 m with subsoiling
-·-·- Q by a drain distance of 40 m with subsoiling
····· Q by a drain distance of 80 m with subsoiling
- - - Q by a drain distance of 10 m without subsoiling

Fig.7. Amount of rainfall N and runoff Q.

In Fig.8 D I you see the variation of the electrical streaming potential on lines 2a, 1b and 5b; on Fig.8 D II the variations of 1a, 3b, and 6b. If you compare this variation with precipitation shown in Fig.7, the function is obvious. The greatest variation shows 1a. On deeper lines a very different kind of variation can be observed, e.g. 4b shows after precipitation a short water movement up to surface and later down: water runs down in great pores and rises in capillaries. The other vertical lines (4a, 3a) show less variation. If we look to 2b, 5a and 6a, we see especially by 6a and 6b about 3-4 hours (2.8) before runoff a positive variation till 5.8 where runoff gets smaller.

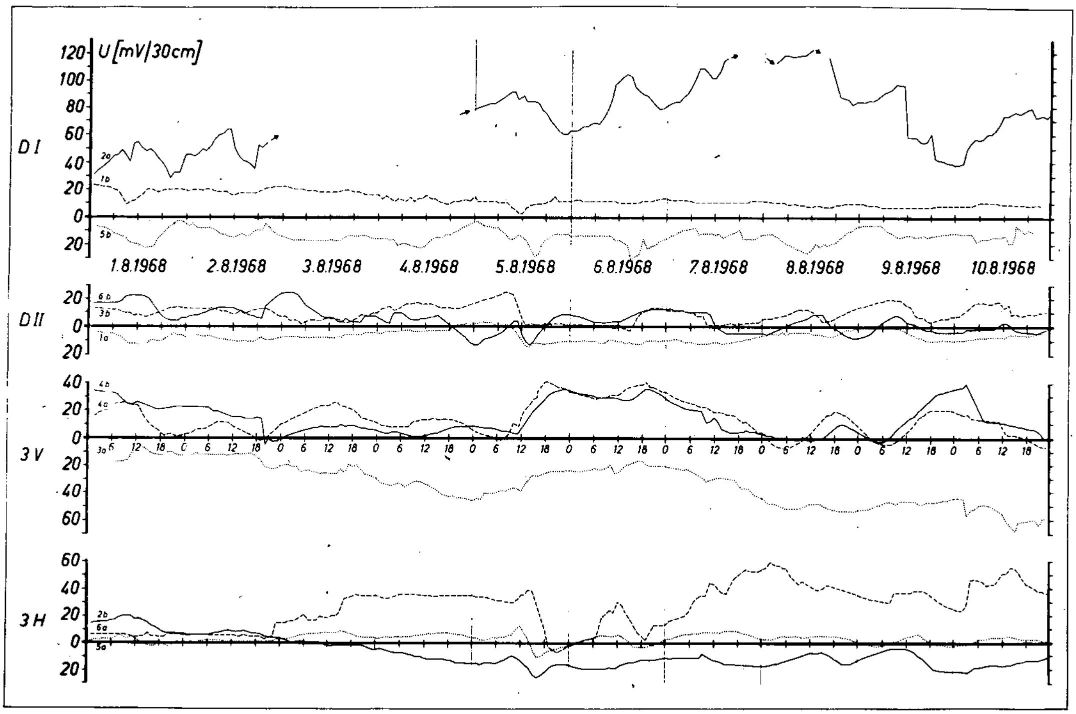


Fig.8. Variation of electric streaming potential on 12 components.

The variation of the amount of vektor $/W_1/$ and $/W_2/$ from D I and D II are shown in Fig.9. The amount of $/W_1/$ is only influenced by the component 2a. $/W_2/$ is rather constant. It is very interesting to see the variation of the angle $\alpha_{1,2}$ and $\beta_{1,2}$. α_1 is the angle between projection of W_1 on plain 1b and 5b and the component 5b. β_1 is the angle between W_1 and 2a. α_2 is the angle between projection W_2 on plain 3b and 6b and the component 6b. β_2 the angle between W_2 and 1a, e.g. α_1 degrees by a great amount of rainfall, the projection of W_1 on 1b and 5b turns to the drain.

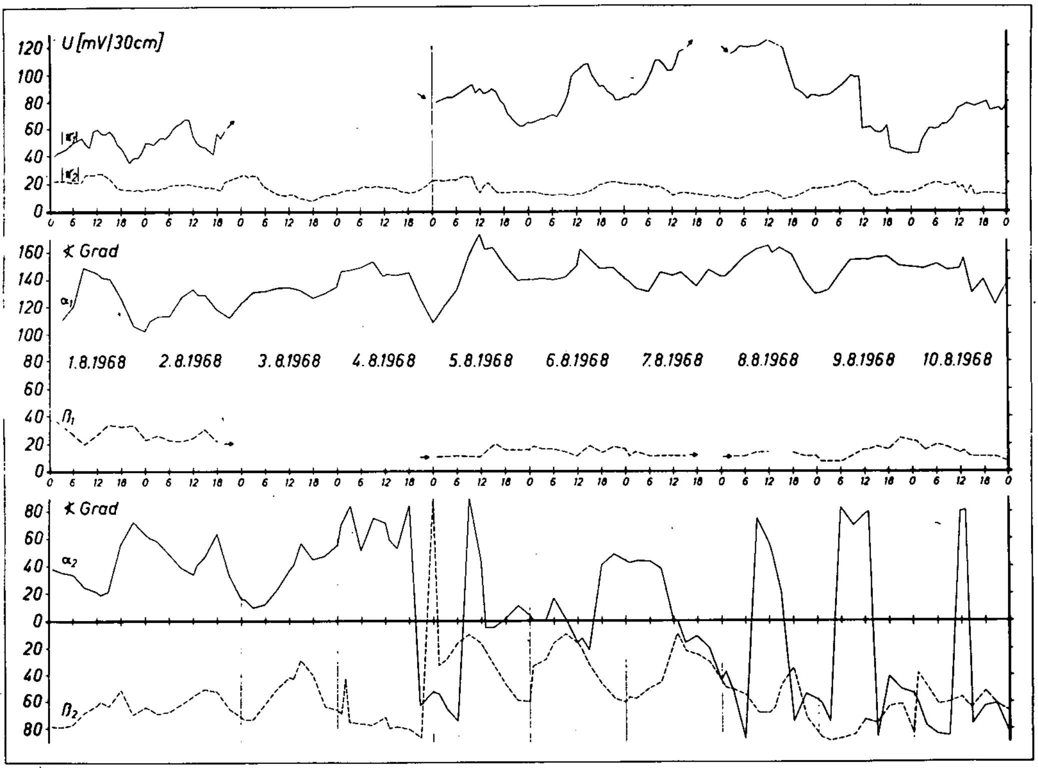


Fig.9. Variation of the amount of /W_1/ and /W_2/ the electric streaming potential and the variation of the angle (= ∢ Grad) $\alpha_{1,2}$ and $\beta_{1,2}$.

If we consider the other value we find that the soil must be very heterogeneous, though the soil is pseudogley of loess. With methods, usual in soil science, the soil seems very homogeneous. We found that water movement follows not the usual expected flow to a drain. If these results are typical for all heavy soils, we think methods of drainage must be changed.

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Paper 1.15

DRAINAGE OF STRUCTURED CLAY SOILS

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Summary

Intensification of the use of grassland on clay soils in the north of The Netherlands makes it necessary to replace the surface drainage system, ridge and furrow system, by subsoil drainage. There is little experience with subsoil drainage; new criteria for subsoil drainage had to be developed for which mathematical or analogue models could be of great help.

The properties of the soil, however, do not fit in the normal framework. It is a structured, non-calcareous clay soil, with cracks from shrinkage which in dry summers go down to one meter and more below surface; in winter these cracks close again.

The flow pattern in this soil is quite different from the pattern usually assumed. The vertical water movement occurs along a few larger pores, such as cracks, rootchannels and wormholes. This was not known when the first attempts were made to replace surface drainage by subsoil drainage.Inadequate sample sizes for laboratory measurements led to the wrong conclusion that the vertical k-sat of a clay layer at a depth between 40 and 80 cm below soil surface was too low to make subsoil drainage possible. Measurements on bigger samples showed that even after a very wet winter there are enough bigger pores, mainly wormholes, to transport the surplus rainfall downward. Also it has been found that subsoil drainage of grassland results in a significant increase of the vertical k-sat of the heavy clay-layer.

Little is known about the unsaturated flow in these soils. Swell and shrinkage may influence the capillary flow, especially in those places where the walls of the soil prisms make contact to each other.

After summer drying, rewetting in autumn developes very slowly, because in the beginning when the groundwater table is low, there is only a small contact area between the surface of the prisms and the infiltrating precipitation. Hence swelling of the soil takes a long time provided that the groundwater table can be maintained low enough to avoid complete rewetting by groundwater.

How drainage may influence swell and shrinkage and thus the unsaturated flow is still an unsolved problem. At the moment it is not yet possible to take this into account in a model testing drainage criteria.

Introduction

A large part of the coastal area of The Netherlands and the north-west of Germany is covered with clay soils. The lighter soils are used as arable land, the heavier, mostly non-calcareous clay is used as grassland.

The grassland is drained by means of ditches and a surface drainage system, the ridge-and-furrow system. Because of the many disadvantages of this system with furrows, several attempts have been made to replace it by a pipe drainage system. For several reasons this never was a success. The intensification of the use of the grassland, however, developed explosively during the last few years. The livestock density rose fom 1.3 to 2.0 cows per hectare and will probably rise to 3.0 in the future. Farmers are willing now to invest in the enlargement of their parcels and the construction of a better drainage system.

The soil types concerned are called Pik, Knip or Knick-clay. In most cases, their profile is built up as follows:

top layer	thickness 20-30 cm; clay content 25-35%; organic matter 5-10%
clay layer	depth 30-70/120 cm below surface, clay content higher than 35%
subsoil	clay content varies considerably from 10-40%

The clay layer has a prismatic structure. The soil prisms have a diameter of 15 cm and a height of 30 cm in average. It was thought that the permeability of the clay layer was low and that this was the main reason for the failure of subsurface drainage. Reported experience and investigations on the subject were rather vague and not convincing.

The drainage problem

An important part of a drainage problem is the formulation of the requirements that the drainage system will have to meet. These design criteria are not a constant, but change with farming practice. Therefore the existing design criteria for subsurface drainage of grassland should be examined carefully and might have to be adjusted to modern, highly intensive farming conditions.

To be able to meet not only present wants, but future demands as well, a simulation model for the drainage process and especially for the vertical moisture flow in the unsaturated zone, could be a great help. Such a model could produce information on the effect of rainfall on the groundwater level and the soil moisture tension of the top layer and thus on the bearing capacity of the soil.

With the help of such a model many questions could be answered, which otherwise have to be studied in experimental fields, which cost much labour and require many years of measuring before reliable data can be given. Experimental fields however cannot be missed. The soil properties in the field do not always correspond to those assumed in simulation models and formulas. Therefore the results derived from simulation have to be checked in the field.

The soil described in this paper is a soil which does not fit into the normal framework. It is a structured, non-calcareous clay soil with vertical cracks from shrinkage, which in dry summers go down to one meter and more below surface. In winter rewetting takes place and the cracks close again.

The flow pattern in this soil is quite different from the flow pattern usually assumed. Untill now some research has been done, about which some information will be given in this paper. This however is not enough to get a good grip on the problem of building a model. This paper therefore is an invitation to come up with suggestions on the best way to go on.

The vertical permeability

It often occurs that the infiltration is hampered by a low vertical permeability due to plough soles, traffic soles and natural compact layers.

In the knip-clay profile, the compact clay layer at a depth from 0.3 to 1.0 m has the lowest permeability. Some years ago several methods for the determination of the permeability have been tested. The resuls are given in

Table 1. Only the results of method 1A were thought reliable and acceptable. Moreover, the results of the other methods were different from the general opinion and did not correspond to the appearance of the area: a landscape with only ditches and furrows. It was concluded that subsurface drainage of knip-clay soils was not possible.

TABLE 1. K-sat values, measured in rings, cylinders and with the auger hole method

Method	Layer (m)	K-sat (cm/day)
I A cylinders		
diameter 0.10 m	0.00 - 0.95	1.9×10^{-4}
height 0.90 m		6.8×10^{-4}
I B cylinders		
diameter 0.06 m	0.23 - 0.53	1.2×10^{-2}
height 0.30 m		2.7×10^{-2}
		8.9×10^{-2}
I C rings		
diameter 0.049 m	0.40	1.4×10^{-6}
height 0.052 m		7.8×10^{-7}
	0.80	1.7×10^{-3}
		4.1×10^{-6}
		1.1×10^{-5}
		1.3×10^{-6}
II Auger-hole	0.30 - 0.60	3.8×10^{-1}
		6.3×10^{-1}
		1.1×10^{-1}
		1.3×10^{-1}
		1.8×10^{-1}
		0.7×10^{-1}

By field experiments with subsursurface drainage of peat soils covered with a clay layer we found that, once the soil was drained properly, the infiltration capacity showed a considerable increase. It could be seen and measured, that the structure of the clay layer became much better within a period of four years.

New experiments were started in the hope that similar results might be reached on knip-clay soils. Because of the disappointing results of measurements at small samples, it was decided to use experimental fields for the investigation of the possibility of sub-surface drainage on knip-clay soils.

In such cases it often is necessary to get information about extremely wet or extremely dry periods. Therefore it usually takes many years to cover a sufficiently longe range of climatic fluctuations. To avoid this problem it was decided that the field experiments should provide the information which is necessary for the building of models with which the effects of extreme weather conditions can be simulated.

One of the first field experiments gave the results of Fig.1. The upper half shows the discharge of the furrows, the precipitation and the groundwater level. The lower half gives the same information about the same experimental plot, but with pipe drains installed midway between the furrows at a depth of one meter. This figure shows that there are no problems with infiltration or percolation after the installation of pipe drains. This experiment proved that small-scale permeability measurements can easily lead to wrong conclusions.

Measured K-sat values

The disappointing results of permeability measurements at small soil samples made it necessary to look for more adequate techniques. Measurements were started using open cylinders with a diameter of 0.3 m and a height of 0.4 m. It appeared that the size of these cylinders was big enough to avoid compaction of the soil and deterioration of the soil structure. The use of these cylinders gave quite satisfactory results, but was rather costly. Therefore the Dutch Soil Survey Institute (STIBOKA) was asked to do some research on the permeability of dense clay layers and to develop a method for the estimation and/or measurement of the permeability which could be applied at a reasonable price in normal soil mapping procedures.

It was expected that the low vertical permeability of the dense clay layer in knip-clay soils would cause most trouble in spring. Bouma, Dekker and Verlinden of the Soil Survey Institute started measurements of the

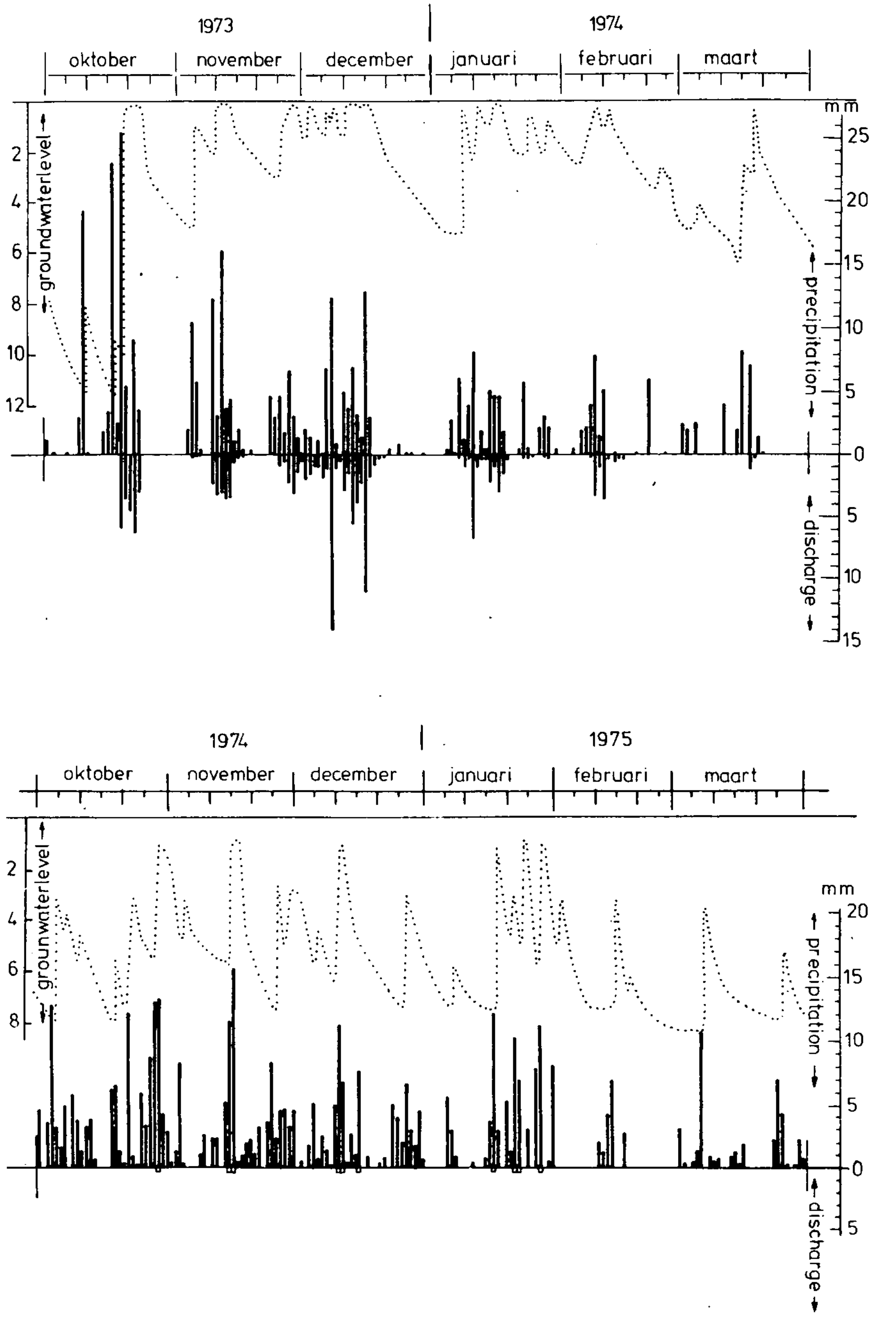


Fig.1. Precipitation, groundwater level and trench discharge in the winter of 1973/1974 (without) and 1974/1975 with subsoil drainage.

vertical permeability in the situation of maximum swell. In early spring they measured K-sat of the clay-layer in situ at thirteen sites. An infiltrometer was placed on top of soil columns (diameter 30 cm, height 30 to 40 cm) from which the sides were enclosed by gypsum, which formed an impermeable boundary to the groundwater flow. First, the masurement was done in situ, subsequently the same procedure was repeated with the soil column detached from the subsoil. This procedure gives an idea of the functioning of the long continuous pores, the wormchannels.

To check if the soil had been measured in the most swollen situation, the columns were taken to the laboratory and saturated with water during a period of three months, after which the measurements were repeated. In the columns taken from grassland, no significant reduction of K-sat values has been found.

The results of these experiments are given in Table 2.

TABLE 2. Vertical K-sat values for the clay layer *

Column	Land use	Drainage conditions	No.of tubular pores		K-sat (cm/day) clay layer		K-sat subsoil
			2-5 mm	>5 mm	attached	detached	(cm/day)
1	grassland	tile drained	11	1	65	430	6
2	grassland	tile drained	17	1	144	500	5
3	grassland	tile drained	14	2	500	500	300
4	grassland	surf.drained	14	3	26	67	41
5	grassland	surf.drained	13	4	53	98	49
6	grassland	surf.drained	31	7	53	116	14
7	grassland	surf.drained	13	–	57	140	4
8	grassland	surf.drained	18	3	57	192	<4
9	grassland	surf.drained	9	4	92	92	25
10	grassland	surf.drained	28	1	47	151	17
11	arable land	tile drained	4	1	12	–	38
12	arable land	tile drained	5	–	13	20	8
13	arable land	tile drained	3	–	18	14	45

* *measured in situ at thirteen locations, in attached and detached soil columns. Also presented are K-sat values for the subsoil (auger-hole method) and the number of tubular pores observed in the top surface of the soil column (BOUMA et al.)*

The flow pattern in structured clay soils

Inside the Soil Survey Institute research has been started on the flow pattern in structured clay soils (Bouma, Dekker, Verlinden, 1976). This study was carried out by spraying a solution of methylene blue on plots in the field of 0.5 by 1.0 m. After that, the whole plot was excavated and all stained walls of pores in the soil were observed and counted. The presence of a stain on the wall of a pore is a clear indication for water movement through that pore.

The following results were achieved:

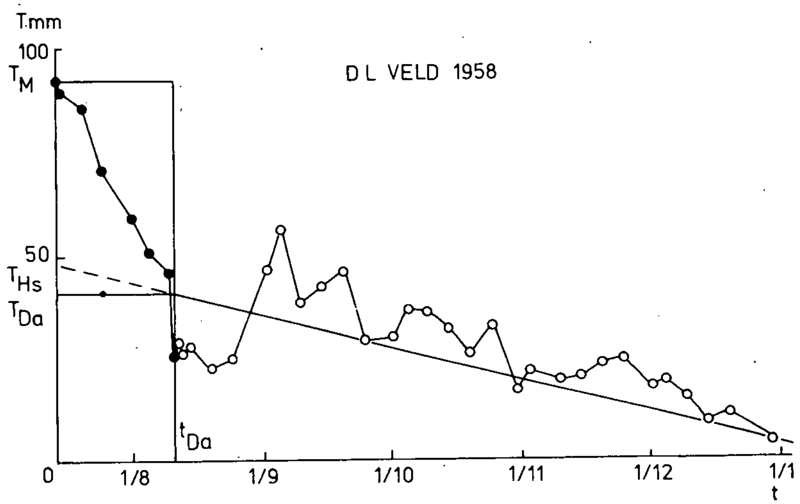
- Mostly the colour pattern consisted of small, generally vertical, separated bands on the walls of voids, separating the peds. The width of these bands was 2.5 - 7.5 mm

- The number of stained bands always was very low. The part of the vertical surface area of the large soil prisms which was occupied by these bands was estimated at almost 2%.

The vertical contact area between the soil and the infiltrating water in the large pores is only small. This will strongly reduce the total volume of lateral infiltration. As long as only a very small part of the total volume of large pores participate in the infiltration process, the capacity to absorb water will be low and practically constant. This probably is the main reason for the fact that rewetting of structured clay soils takes a long time.

Rewetting of structured clay soils

The rewetting of structured clay soils has been studied by Makking and van Heemst (1965). Fig.2 shows the process of rewetting. For the calculation of the soil moisture deficits, the highest moisture content at field capacity (deficit = 0) measured during the period 1952-1960, is taken as a reference level. After the soil has been dried out in summer, rewetting takes a long time. The authors found that after the moisture content of the investigated soil was instantly brought back to field capacity and the soil was optimally supplied with water; it took 178 days before the soil moisture content had reached the original highest level.



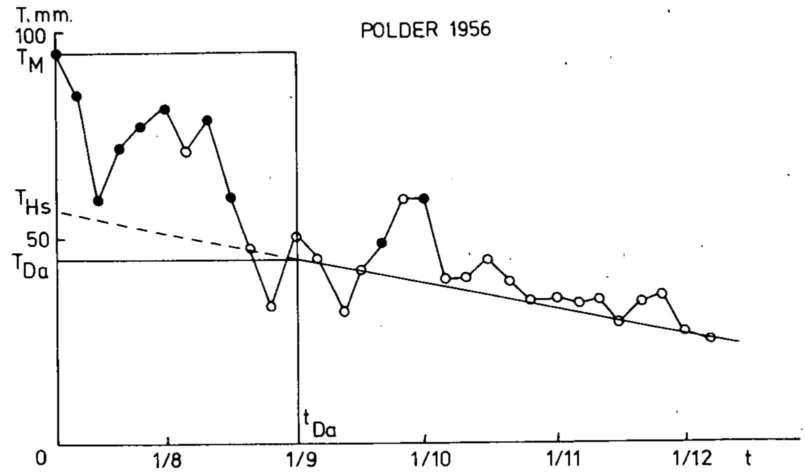


Fig. 2. Moisture deficits since the moment of the largest deficit in summer (T_M) in a drainage lysimeter field (DLV) and a polder in 1956.

• situation without drains   o situation with drains

The smooth straight line represents the decreasing deficit at field capacity. T_{Da} is the deficit at the first appearance of drainage water.

(After Makking and van Heemst, 1965).

The process of rewetting has also been studied at the Institute for Land and Water Management Research (ICW). In the laboratory research has been done on the capacity to absorb water of the prisms taken from the clay layer of the knip-clay profile (du Bois). The prisms have a height of 0.3 m and a diameter of 0.15 in average. The clay content is 50%, the organic matter content less than 10%. With sprinklers the whole surface of the prisms was wetted. This is quite different from the situation in the field. Bouma et al. found that only 2% of the available vertical surface area of the large soil prisms was wetted after sprinkling. Nevertheless such experiments do help to understand the process in natural conditions.

The results are shown in Fig.3. Similar results have been found by Bouma, Dekker and Wösten (1978).

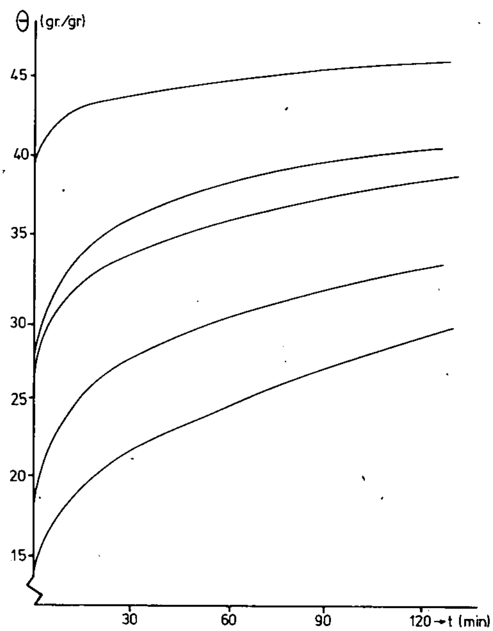


Fig.3. The increase of moisture content (g moisture/100 g dry soil) due to sprinkling 30 mm/h on samples with a different moisture content at the beginning (after du Bois).

Swell and shrinkage

The same clay soil was used to measure swell and shrinkage. Samples with heights of 2 and 5 cm were attached to a plate in which the moisture tension could be set to values varying from 30 to 135 cm H_2O. In all cases except the measurement at 70 cm H_2O, there was a good contact between the soil prism and the plate. Here again the conditions of the experiments differed much from those in the field. The results however supply useful information (Fig.4).

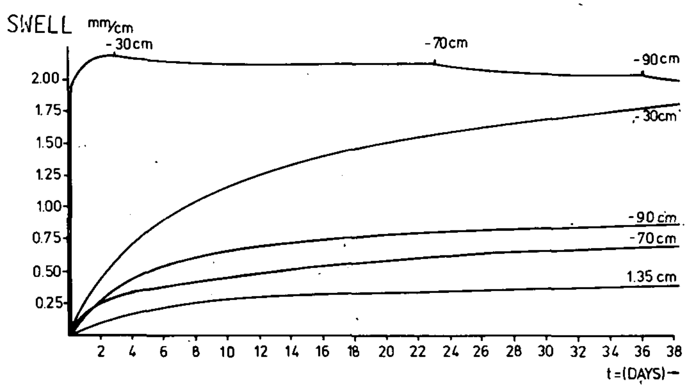


Fig.4. The upperline gives the swell after two days with a moisture tension of the ceramic plate of 0 cm H_2O and hence the shrinkage due to the tensions of -30, -70 and -90 cm H_2O.

The other lines give the swell due to -30, -70, -90 and -135 cm H_2O (after du Bois).

Discussion

It was feared that the vertical permeability of the soil concerned would not be high enough to make a subsurface drainage system work properly. But after a closer look into the matter there seems to be no problem. The experiments give rise to the hope that by subsurface drainage the process of rewetting can be retarded, that swell will develop slowly, that the large pores will stay intact during nearly the whole winter period and that the vertical permeability will constantly be at a good level.

We expect that the replacing of the surface drainage system by a subsurface drainage system will cause no problems. For the farmers it will be a big advantage to be freed from the furrows in their parcels.

A good subsurface drainage system however can offer more. The farmers need soils with a sufficiently high bearing capacity for their cattle and machinery. As to the bearing capacity, which depends on the moisture content of the top soil, there are two critical periods, autumn and spring. Moisture contents stay low a long time after summer and even a part of the winter if the groundwater can be kept at a sufficiently low level. The main problem is now to find out how long and with what frequency the groundwater can be allowed to exceed a certain level. Before simulation models can help to answer questions about drain distance and drain depth, the complex relation groundwater-rewetting-bearing capacity needs more study.

In spring when the moisture content of the top soil has reached its maximum value, drainage must help to decrease the moisture content of the topsoil in order to increase the bearing capacity. In fact the moisture content of the top soil is decreased by the process of evapotranspiration, but the drainage is needed to keep the groundwater table at a sufficiently low level, to reduce capillary rise. Swelling and shrinkage may influence the capillary flow, especially at those places where the walls of the soil prisms make contact to each other.

Little is known about the unsaturated flow in these cracked soils. Calculations with simulation models for the vertical, unsaturated moisture flow in structured clay soils are still not possible. Nevertheless it is worthwhile working at it, for until now, it is one of the best tools to handle drainage problems.

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Group 200

Drainage materials

CONTENTS

Paper 2.01

DRAIN ENVELOPE MATERIALS IN CANADA

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Summary

Envelope materials have always been used with problem soils in Canada. Originally this was grass sod, straw or tar paper. About 1957 thin synthetic envelope materials became available and field and laboratory tests were conducted periodically to evaluate their use. The results of these tests have been printed. No failures have been reported in the field where envelope materials have been properly applied. In 1977 about 10% of all drain pipe installed used an envelope. The material is a polyester knitted fabric sock.

Introduction

Drains were installed in Canada by trenching machines for nearly half a century prior to 1950. There were few reported cases of drain failure due to silting. Only the land where drainage could be achieved easily was drained at this time, usually with wheel-type trenching machines. Fields only were drained, not entire farms.

The following decade was transitional. Farming began to change. Corn for both grain and silage replaced much of the small grains, hay and pasture. Table 1 shows the huge increase in corn production in Ontario. Corn demands good soil drainage. There was also a change from dairying to cash cropping.

TABLE 1. Area of land used to produce corn, Ontario, Canada

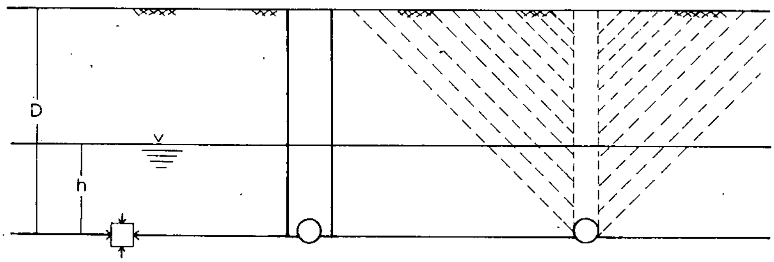
Year	Grain corn (ha)	Fodder corn (ha)	Total (ha)
1950	113,805	138,510	252.315
1955	228,825	110,970	339,795
1960	182,250	114,210	296,460
1965	299,700	170,100	469,800
1970	445,500	228,825	674,325
1975	575,100	295,650	870,750
1977	652,050	350,330	1,002,380

By 1970 the drainage industry was also undergoing a corresponding change to cope with the demand for improved farm drainage. The introduction of faster chain-type and plow-type drainage machines, corrugated plastic drainage tubing, laser grade control, and synthetic envelope materials all occurred in a five-year period. Contractors could not wait until researchers examined the problems, and many of the problems are not yet solved.

Typical field situation

Drains are usually installed at a depth of 0.7 to 1.0 m in Ontario. Failure of drains due to silting is a frequent problem with non-cohesive soils, and in areas with a high water table at the time of installation (spring or late fall, for example). A wheel or chain type trencher mills the soil producing irreversible structural changes to any existing soil structure. When using standard methods of backfilling with wet soil into a wet trench, the density of the backfill is about 15% less than that of the parent material. Irwin (1971) showed that when a plow-type drainage machine is used to place drain tubing, the soil density above the drain tubing is also about 15% less than the parent material (see Fig.1).

Critical hydraulic gradient is the gradient at which the buoyant weight of a volume element is balanced by the vertical component of the body force due to water flow into the drain. No erosion is assumed to take place until the gradient is exceeded by the vertical component of the exit gradient at the drain. Failure can take two forms - erosion through grain migration of a



(a)	(b)		
1.55	1.25	dry bulk density (ρ_d)	g/cm^3
1.97	1.78	wet bulk density (ρ)	g/cm^3
0.71	1.12	void ratio (e)	
27.00	42.00	water content (e) %	
0.96	0.78	critical gradient (i_c)	

Fig.1. *Soil cross-section illustrating*

(a) a soil element at the base level of a drain
(b) drain installed by trenching machine, and
(c) drain installed by drainage plow.

Typical values are D = 0.8 m, h = 0.5 m, particle density = 2.65 g/cm^3, and others as tabled above.

small volume due to flow concentration into the drain pipe, or, the bulk heave of a large volume such as the trench bottom which may put the drain off grade. Heave without boiling results in an increase in void ratio and consequent increase in permeability and possible failure through boiling. Schmidbauer (1950) found that bulk heave occurred if a sand contained more than 10% by mass of grains finer than 40 µm, and if the hydraulic gradient was sufficient to induce failure.

The total stress, σ, at the base of the drain for the example in Fig.1 (a) is:

$$\sigma = \bar{\sigma} + u$$

where u is the neutral stress equal to $\rho_w h$ and $\bar{\sigma}$, the effective stress equal to the mass of soil and water above the drains base. The total stress can be called the surcharge. Surcharge gives the soil a confining strength which may reduce soil instability.

When the effective stress equals zero a quick condition exists in the soil. This situation is particularly common with fine sands of uniform particle size in a loose or open state of packing.

Small amounts of clay or silt give the soil cohesive properties which adds to its strength. Cohesion may range for the effective stress condition to about 2000 kg/m^2 which exceeds hydrodynamic seepage forces. The pressure of cohesion reduces soil instability. The plasticity index of a soil should be determined to evaluate this important soil property.

The critical hydraulic gradient, i_c, for the unconfined condition is:

$$i_c = \frac{G - 1}{1 + e}$$

where G is the particle density of the solid material and e is the void ratio. This term is independant of the soil hydraulic conductivity, velocity of flow and particle diameter. For sand with a particle density of 2.65 and a void ratio of 0.65 the critical hydraulic gradient is unity. The general equation for instability at a subsurface drain is:

$$i_c = \frac{G - 1}{1 + e} + \text{surcharge} + \text{cohesion}$$

The hydraulic conductivity, k, is related to void ratio, e, thus:

$$k = c_v \, m_v \, \gamma_w = c_v \frac{a}{1 + e} \gamma_w$$

where

c_v is the coefficient of consolidation

m_v is the coefficient of compressibility

a is the coefficient of volume compressibility

After installation of a drain the backfill is very loose (Fig.1b), or the parent material is loosened by the drainage plow (Fig.1c), creating a higher void ratio and bulk volumetric change above the base of the drain.

Darcy's law governs the flow to the drain. Assuming the hydraulic gradient to be unity and unchanged in the intitial stages after drain installation the higher void ratio permits a higher hydraulic conductivity. The increased velocity of flow detaches soil particles and carries them into the drain. This was observed by Hore and Tiwari (1962) who determined that the soil entering the drain came from the backfill and not the parent material. Tests at Guelph showed that soil will move into unprotected drains with heads as low as 25 mm.

Synthetic envelope materials

When properly installed, most synthetic envelope materials used in Canada adequately perform the functions of passing water into the drain while excluding sand.

A glass fibre filter with parallel reinforcement (manufactured by L.O.F. Glass Fibers Co., Toledo, Ohio) was used in Ohio in 1956. About 1957, the same product was marketed in Canada by Globe Glass Saturaters, Petrolia, Ontario. The material tore easily, and in 1959 "Tile Guard S-110" with random reinforcement was introduced. The cost was 5¢/m. It was marketed in rolls of 365 m. The rolls were attached to the sand box of the trenching machine and mechanically placed above and below the drain pipe. Tile Guard is an inert lime borosilicate glass filament held together with phenol-formaldehyde binding agent.

"Durant type 204" was marketed at the same time for use as a stable base under the drain pipe in unstable soils. It is a glass fibre reinforced material saturated with bitumen. The price was 3¢/m.

About 30,000 m/yr of these products were used in Canada from 1957. They were satisfactory as long as the contractor took care to wrap the pipe adequately. Polyethylene sheet underlay was often substituted for Duramat. However, these products proved to be difficult to adapt to the plow-type machine they ruptured if the plow stopped and then started again. "Tile Guard PG-90" was introduced to overcome this problem. High rate machine operation made the application of this form of protection difficult and some failures occurred where the upper and lower rolls failed to meet properly. Tile Guard has remained the standard product where clay drain pipe is used.

In 1973 the Big O Drain Tile Company, Hensall, Ontario, installed a small knitting machine to produce a sock for pre-wrapped plastic drain pipe. At the same time they marketed Drain-O-Guard, the Cerex spun-bonded nylon sock patented by Advanced Drainage Systems, Inc., in the U.S.A. Cerex was satisfactory as a filter but was hard to get into the ground without tearing. Each roll was delivered in a plastic bag for protection. The cost of the envelope was 23¢/m. In 1975, the Big O Drain Tile Co. started to manufacture nylon knit sock which was more resistant to abrasion and damage in transport and in field handling. In late 1973, other manufacturers adopted Remay polyester (14 g/m^2) and continued to increase its thickness. When 36 g/m^2 proved unsatisfactory, these companies also changed to a knitted sock. A knitted sock is now the standard form of protection for most of the plastic drainage pipe used in Canada. The contractor does not charge extra for installation of this type of pre-wrapped pipe. The cost of the envelope is 26.2¢/m in addition to the 56¢/m for 100 mm pipe.

In 1977 about 10% of all corrugated plastic drainage tubing was shipped with a pre-wrapped envelope. Some contractors will use this on 40% of their work.

Problem soils

The soils in Ontario and Quebec which tend to cause silting of drains were developed from deltaic medium to fine sands and silts deposited on a till plain. The topography is nearly level. The pervious upper sand strata over an impervious clay layer typically at 1 to 2 m depth contributes to a naturally waterlogged condition. Until recently, suitable drainage outlets were not available for many of these areas.

Complete information on these problem soils is not available. Fig.2 shows the range of the grain size distributions for these problem soils. Most drain failures occur when significant proportions of the soil grain sizes are between 50 to 120 μm in diameter.

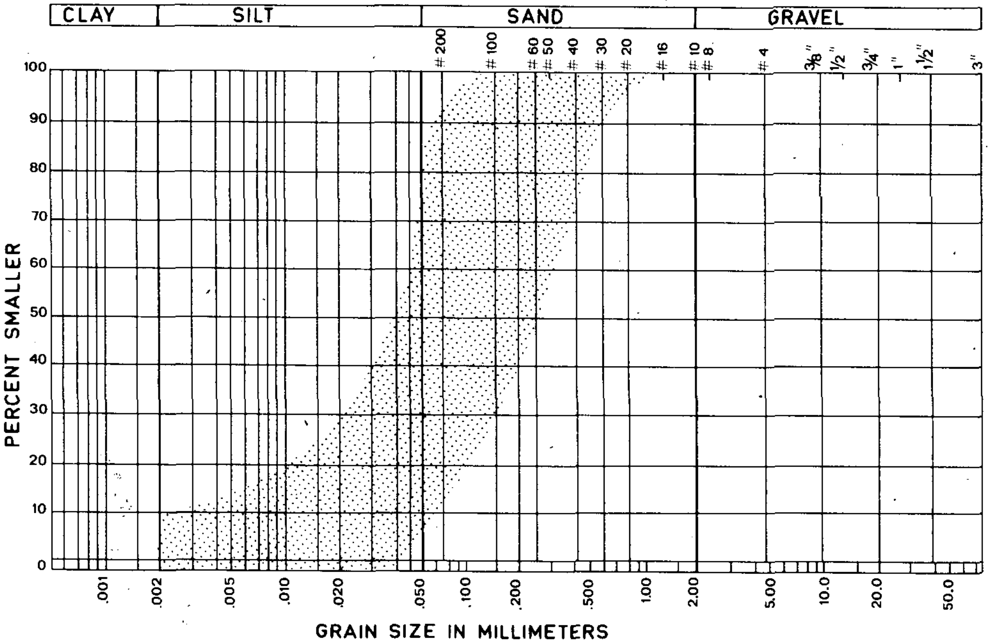


Fig.2. Range of grain size distribution for problem soils.

Canadian research on envelope material

In 1957, a laboratory study was initiated by the senior author on the relative merits of seven types of protective materials used to overcome the problem of drain silting. This study was prompted by the silting-up of 16,000 m of drains on the Horticultural Experiment Station, Vineland, Ontario. Evaluated were blinding with top soil, Duramat cover, Tile Guard cover, Tile Guard above and below, Kraft impregnated building paper cover, No. 2 saturated tar paper cover, and straw cover. Tile Guard above and below provided a tenfold increase in flow and protection from silting compared to the other treatments.

In 1959, Hore and Tiwari (1962) tested the following treatments in the laboratory using Granby sandy loam: blinding with top soil, Tile Guard above and below the drain, Duramat above the drain, and Tile Guard above - Duramat below the drain. Rainfall on the backfill material followed by groundwater flow to the drain was simulated in 6 hour tests. Tile Guard above and below allowed maximum flow with minimum soil entry. Sand was transported into the drain by water draining from the backfill, not from the parent material.

In the fall of 1960, the authors commenced a 3.25 ha field experiment near Lancaster, Ontario, in Bainsville silt loam where drain failures had occurred. Five treatments (three replications) were evaluated: blinding with top soil, straw cover (3 kg/m), Tile Guard above and below, tar paper above, and Tile Guard above - Duramat below. The drains were 122 m long laid at a grade of 0.0017 with sediment sampling points at 30, 61 and 91 m from the outlet end. The hand installation of materials was less than ideal due to torn filters and some uncertainty of proper placement. However, statistical analyses of four samplings over an eight year period ranked the treatments in decreasing order of effectiveness as follows:

> Tile Guard above and below, Tile Guard above - Duramat below and straw cover (equally effective), blinding with top soil, and tar paper above. An additional analysis of variance involving "Linear Contrasts" showed that complete cover treatments were more effective than top cover treatments (tar paper and straw) and blinding with top soil.

In 1973 several additional materials were introduced by manufacturers and many problems developed such as tearing and general concern for effectiveness. The drag force was therefore determined by the senior author by pulling a known area of material over a wetted galvanized sheet of metal. The results are given in Table 2. The dry breaking strength of the Tile Guard S-110 was 470 N/m^2 but only 294 N/m^2 when wet. The material ruptured in the field under wet conditions.

TABLE 2. Physical drag force of envelope materials

Envelope material	Drag force (wet) N/m^2
Tile Guard, S-110	196
Tile Guard, PG-90	333
Globe Glass - nylon felt	314
Big O Nylon Weave 405 sock	stretched
Drain-O-Guard (Cerex)	461
Big O new sock	stretched

The above materials were also tested for effectiveness of filter action and for flow rate. The tests were made with a constant head permeameter fitted with a slotted plate. The slots were cut the same size as the perforations in commercial corrugated plastic tubing. Only very small quantities of soil passed through any filter; therefore each filter performed satisfactorily.

For the same hydraulic gradient, the flow rate through the soil and the soil plus filter was the same; therefore, the envelope materials did not affect the flow rate.

McKyes and Broughton (1974) found in the laboratory that full wrap glass fibre sheet and polyester weave sock filters provided superior performance to jute and hemp twine wrapped in the grooves of corrugated plastic drainage tubing. The latter treatments became plugged with fine sand resulting in unacceptable low flow rates after 10 to 20 days.

Rapp and Riaz (1975) in a laboratory study using a gravel filter and glass fibre filter material combinations similar to those studied by Hore & Tiwari (1962) verified that completely wrapped treatments other than the gravel filter provided the best protection. The gravel treatment provided the best flow characteristics but the poorest protection from siltation.

Broughton et al (1976) tested in the laboratory most commercially available materials in North America and showed that drainage rates decreased with time; however, they recovered temporarily after a period of no flow, a condition frequently found in the field. They concluded that any full wrap material except coconut fibre would do a good job of excluding sand from a drain and permitting water to enter. These studies are being followed up by a field study of seven treatments installed in August 1976, in a Nicolet sand soil where unprotected drains have previously failed.

Pore size distribution of envelope materials have been determined by several techniques. Nelson (1960) used a dry sand sieve technique to measure the pore size distribution of thin envelopes. Miano (1977) used a Ballotinin ball technique. Suction methods have been adopted in The Netherlands for thick envelopes. Other methods in common use include that of mercury immersion (A.S.T.M., 1974). Most of the above techniques are laborious.

The Quantimet 720 has been used to determine pore size distributions of geological and soil materials (Ismail, 1975; Murphy, 1977). This instrument is an image analyzing computer and is used to measure in two dimensions the number and the area of voids in a thin section of material.

The drain envelope materials used in Canada are about 0.15 mm thick and can be assumed with reasonable accuracy to be two dimensional with respect to voids. Cerex spun nylon, Tile Guard S-110, Tile Guard PC-90, and a coarse and fine mesh nylon fabric sock as currently used with 100 mm corrugated plastic drainage pipe were analyzed in January 1978 by the Quantimet 720[1]. The results of these analysis are shown in Fig.3.

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[1] *The Quantimet 720 manufactured by Imanco, Royston, England was operated by Dr W.Petruk, Mineral Sciences Laboratory, Dept. of Energy, Mines and Resources, Ottawa, whose skilled assistance is gratefully acknowledged.*

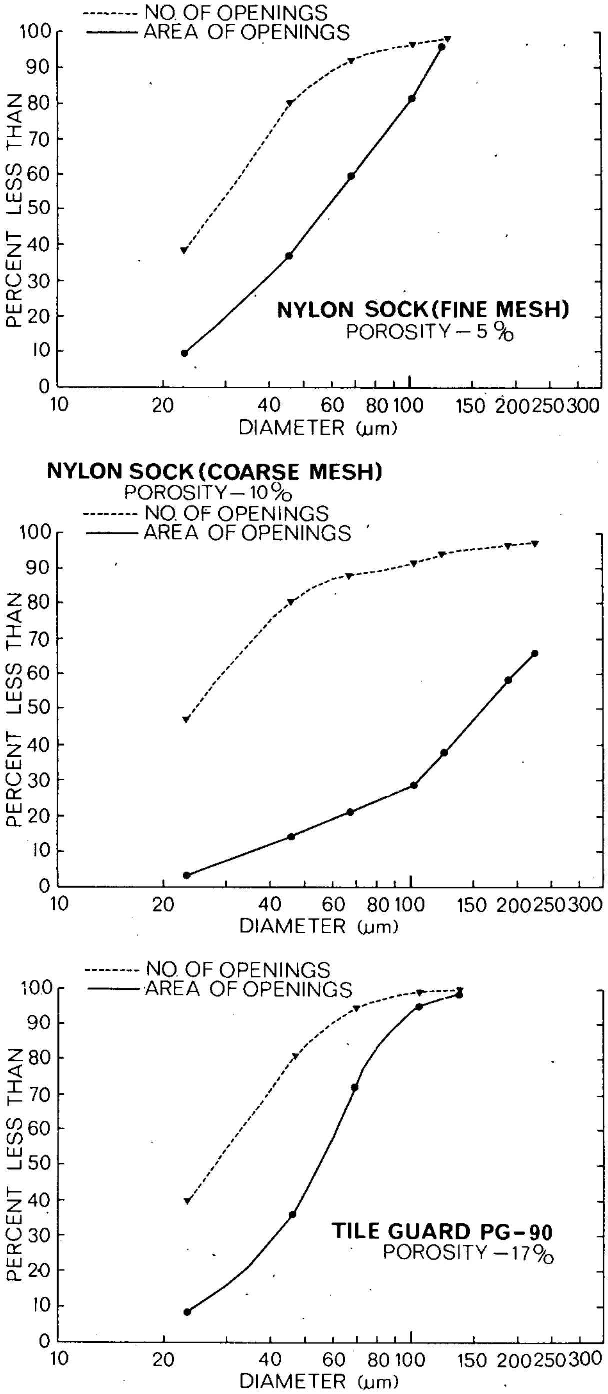

Fig. 3. Pore size distribution by the Quantimet computer for five filter materials.

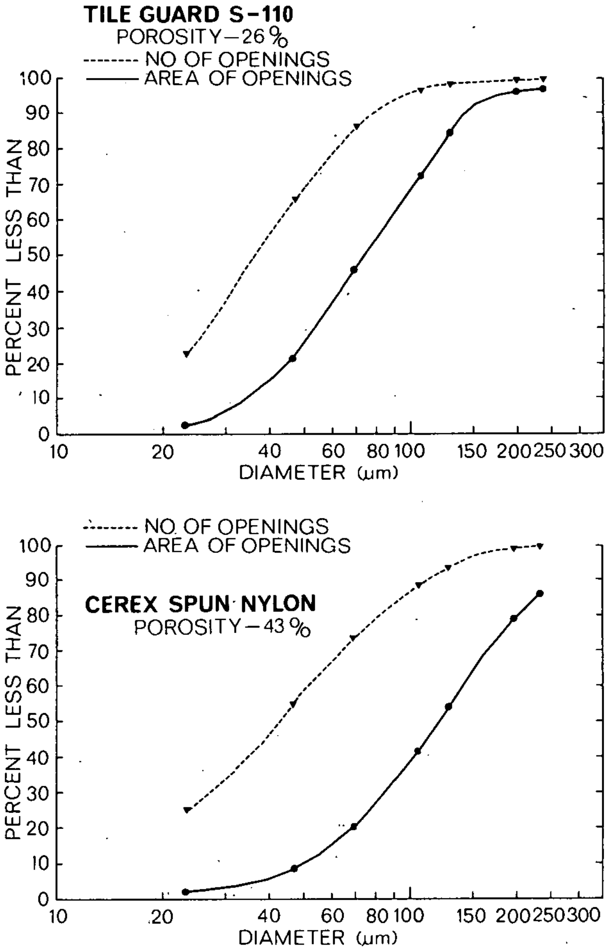

Fig. 3. (cont.)

Figure 3 shows for each envelope material the relationship between percent less than and opening diameter as derived from the measured area of openings. It also shows percent less than, and the measured number of openings of different diameters. Total porosity for each material was also computed from the ratio of openings to the total area analyzed. The results of these analyses (Fig. 3) showed that these filters have a wide range in total porosity (5 to 43%) and a wide range in uniformity coefficients. The Uniformity Coefficient (D_{60}/D_{10}), based on area of openings, varied from 2.4 for Tile Guard PG-90 to 5.6 for the coarse mesh polyester sock. Both glass fibre products, Tile Guard PG-90 and S-110, had similar distribution characteristics, but the pore sizes of PG-90 were generally smaller. In all cases, the analyses based on the measured number of the various sized openings gave a finer pore size distribution than that based on the measured area of the openings.

Opinion and questions on the state of the art

The problem of drain silting is understood. The flow conditions which cause the movement of solid material is known. The physical properties of the soil can be measured. The physical properties of the envelope materials can also be measured. How do we put this information together to forecast in advance of construction which soils require protective envelopes? In cohesive soils, do envelopes cause drain failure?

An envelope adds 23% to the cost of a drainage system; therefore, envelopes should not be used unless required.

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Paper 2.02

SYNTHETIC DRAIN ENVELOPE MATERIALS

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Summary

Synthetic drain envelopes are suitable for use in protecting subsurface drains providing the hydraulic failure gradient of the soil is not exceeded in the installation. The problems of development of suitable specifications for synthetic envelope use remain. As new materials become available, standards will be necessary for evaluation of their utility.

Introduction

Whenever a buried subsurface drain is to be installed, one of the questions that arises is the necessity of including in the installation some measure to protect the drain from soil particle entry. This protection has in the past been called the drain filter. The concept is that placement of this porous material around the drain will filter out the soil particles allowing only clean water to enter the drainage system, thus preventing problems of sedimentation and reduction of drain capacity. There are no established criteria for determining when drain filters are necessary and when they are not required. At the present time most drains installed in humid areas are installed without filters, and drains in arid areas are installed with filters. The practice is followed almost universally without regard to soil type or condition. As the functioning of the material around drains has become better understood, it is obvious that it should be called a drain envelope rather than a filter. The nature of a filter is such that it removes and retains material, becoming clogged in the process. A drain envelope must protect the drain for an indefinite period without losing its desirable characteristics.

The placement of special material around drains began in humid areas. Drain trenches were prepared, the drains were placed in the bottom of the trench and in order to maintain the alignment of the drains, topsoil material was tumbled down onto the surface of the pipe. The purpose of this practice was to maintain the alignment of the pipe until the trench could be backfilled. In the United States this practice is referred to as blinding. Most drains in humid areas installed this way functioned very well and were successful over a long period of time. The blinding material in humid areas was the topsoil which is highly aggregated and is hydraulically very stable.

When subsurface drainage began in the West, a similar installation practice was followed. The trench was dug, the pipe was placed in the trench, and material from the soil profile was tumbled onto the pipe to maintain the alignment until the trench could be backfilled. Many arid region soils were dry and very hard. In addition, the soils were alluvial and many of the profiles contained layers of sand. The workers who were assigned the responsibility of covering the pipe found that sand lower in the trench was easy to use to cover the pipe and so rather than use surface soil, the sand from the profile was placed around the pipe. Experiences soon showed that the most successful drains were those that had been covered with course sand material It was also discovered that drains completely surrounded by the granular sandy material worked best. It was then determined that this sandy material was more desirable and, therefore, should be imported where it did not exist naturally in the profile. The practice of using drain envelopes then became established in arid region drainage.

In humid areas, drains were originally placed in a random pattern. The drains followed low areas which had previously been wet and in which the surface soils were highly developed and structurally stable. The surface soils used for blinding made excellent envelope materials. As humid area drainage became more industrialized, a grid pattern of installation was followed and drains passed through many soil types including very fine sands. When fine sandy soils were encountered, drain failures followed. Placement of the blinding material over the tops of the pipes did not adequately protect the drains from sediment entry. It soon became obvious that under some soil conditions even in humid areas envelope material must completely surround the drains.

Drain envelope design criteria

Development of the idea of filter or envelope materials for drains followed the development of a reverse filter for protection of earth-fill dams from piping. This material was called a reverse filter and the nomenclature of filter has carried over into the use of this protective material around drains. The design of a reverse filter for the protection of a dam requires selecting sizes of uniform material such that the openings in one material will not permit entrance of the next size material. By beginning with fine material with the proper opening size with respect to the soil and adding successively coarser layers, a system can be made which will allow water to flow sediment-free out of the soil. The complications of installation prevent reverse filters from being used around drains. The cost is too high and the installation is slow and complicated. As a result, most of the attention related to envelope material for drains has been on the grading characteristics of the material with respect to the grading characteristics of the soil. The U.S. Army Corps of Engineers and the U.S. Bureau of Reclamation have made extensive studies of the problem. The result has been a set of specifications for graded gravel envelope materials for protection of drains. These specifications have been used successfully for a number of years by both the U.S. Soil Conversation Service (1973) and the U.S. Bureau of Reclamation (1978). The drain envelope specifications are written in such a way that a range of natural materials will serve satisfactorily as envelope materials. The design process results in a graded material that is suitable for use with a particular soil. The evaluation of suitability is accomplished by making a mechanical analysis on the soil or base material and making a mechanical analysis of the proposed envelope materials and then checking whether the envelope materials fall within the desirable range of sizes allowed by the specification.

The function of a drain envelope is not clearly understood. This fact is evidenced by the fact that drain envelopes are alternatively referred to as drain filters. The U.S. Soil Conversation Service (1973), for example, distinguishes between gravel envelope materials placed around drains to provide bedding and to increase permeability adjacent to the drain openings, and similar material placed around drains to prevent sediment inflow. Strict grading specifications are required for filter materials. Envelope materials have a wider range of allowable sizes and grading. Both filters and envelopes, however, have the same minimum limitation of not more than 10 per cent passing the No. 60 sieve (0.25 mm).

The U.S. Bureau of Reclamation (1970) considers gravel envelopes to be a part of the complete drainage system. The Bureau has developed their own gravel envelope grading specifications emphasizing both prevention of sediment inflow and high permeability to reduce inflow resistance. By examining the grading curve of the soil material at drain depth, an allowable set of grading limits for the envelope material is selected. Fine material in the envelope is restricted to less than 5 per cent passing the No. 50 (0.30 mm) screen. In addition, the coefficients of uniformity and curvature of the material must fall within specified limites. An additional requirement is made that the measured permeability of the envelope material must exceed the permeability of any material along the length of the drain. The specification for required envelope permeability is based on a theoretical development by Moody for headloss in the vicinity of a circumferential drain opening. The required envelope permeability is a function of joint width, pipe radius, envelope thickness, pipe length, the permeability of the soil and the flow expected into the joint or drain opening. Use of these specifications has resulted in a high degree of success in extensive field installations.

Both the envelope and filter concepts for drain envelopes have functioned successfully. The relatively small differences in the two criteria do not seem to cause significant differences in drain envelope performance. Failures in both cases have been related to an excess of fine soil particles in the envelope material either due to faulty quality control or due to soil material being incorporated into the envelope material during construction.

Some drain envelope failures have occurred where single size large aggregates have been used in unstable soils.

Drain envelope functions

The adaptation of the concept of a reverse filter for dam protection to protection of subsurface drains from sediment inflow has led to the common use of the word "filter" for the material placed around the drain. However, this material serves a number of other purposes in addition to preventing the inflow of sediment. For this reason it is preferable to describe the material as an envelope around the drain. A drain envelope restricts sediment inflow and provides material of a high structural stability close to the drain. This in turn improves the hydraulic conductivity immediately adjacent to the drain and also provides suitable bedding material for the drain tube. The problem of referring to a drain envelope as a drain filter prevents adequate evaluation of the type of material necessary for use to protect the drain from sedimentation. As stated previously a filter by its very nature becomes clogged in performing a filtering action during a given period of time. A drain envelope placed around a drain should not clog with time or it loses its effectiveness, therefore, filters should not be placed around drains. Drains should be protected with drain envelope material.

The advent of lightweight plastic drain tubing and the convenience of installation provided by the tubing has led to a search for a lightweight substitute for gravel envelope material that is commonly used around subsurface drains. Supplies of suitable natural materials are scarce or nonexistent in many areas, and in some cases constitute the principal cost of drain installation. These conditions have encouraged a search for a gravel envelope substitute in the form of synthetic fibre materials.

Synthetic envelope materials

The first synthetic envelope material extensively used in the field was made of fibreglass. Those investigating the material considered the drain envelope to be a filter and when low cost fibreglass fabric became available it was used as a substitute for the gravel material around the drain. Two problems occurred. The glass fibres, being very fine, had a large surface area exposed to chemical action in the soil. The first types of fibres used dissolved in the soil in a very short time. Research quickly discovered that Pyrex type glass was resistant to dissolution in the soil and so use of borosilicate glass became necessary. The second problem was that fibreglass envelope materials rapidly became plugged with soil particles. The materials performed very well as a filter and the limited open area where water passed through the fibreglass and entered the pipe was soon plugged. The problems that affected fibreglass must also be taken into consideration in the design of envelopes made of synthetic fibres.

Other synthetic envelope materials which have been tried have included organic products. Although straw is not an artificial material, it is an artificial envelope material when considered for mineral soils. The problem with the use of straw was that it deteriorated rapidly in the soil and the effectiveness of the envelope was soon lost. A natural vegetable material which is more stable is coir or coconut fibre. As synthetic fibres and rubber replaced coconut fibre for carpet padding, coconut fibre became availabale for use around drains. However, the material was relatively expensive to use. Attempts were made to incorporate the coir fibre wrapping on the pipe as part of the pipe manufacturing process to reduce the installation cost. The material seemed to function satisfactorily in the field, but has not been used extensively. Only a few experimental installations using coir have been made in the United States.

The most readily available materials in large supply at the present time are synthetic fibres of nylon, polyethelene, and polypropylene. These materials can be spun into fine fibres and formed into bonded random fibre mats at a relatively low cost. The thickness of the material can be controlled and the size of the openings can also be controlled either by the density of the material or the thickness. Most of the synthetic envelope materials proposed for use in the United States are random fibre and spun-bonded.

Woven synthetic fibres have also been proposed for use as drain envelope materials or in soil stabilization situations where they are intended to be used as a permeable membrane. The biggest market and use of these materials at the present time is in civil engineering construction where they are used to stabilize road beds and slopes beneath large stone covers used for bank protection. All of these fabrics, whether woven or random fibre, have a potential use as synthetic envelope materials to protect drains.

Tests for synthetic envelope materials

Tensile strength is one of the important physical properties of a synthetic drain envelope material. The material must be strong enough to bridge the corrugations of plastic drain tubing without tearing or sagging deeply into the openings. It must also have sufficient strength to withstand the stresses of construction and shipping as well as stresses occurring as it is being applied to the pipe.

Abrasion resistance for synthetic envelope material is important because rolls or coils of pipe and lengths of pipe are dragged over the ground at times and the pipe must also pass through guides in the drainage machines during the time of installation. The material without abrasion resistance would develop large holes which would render the envelope material ineffective for restricting soil movement into drains.

Chemical resistance of synthetic envelope materials is also important. The very fine fibres in sythetic envelope materials have a very large surface area exposed to chemical reactions in the soil. The pH of the soil may be very high under sodic soil conditions and may be very low during cleaning operations which included the use of sulphorous acid for removal of iron ochre and manganese deposits in drains. The resistance of these fine fibre materials to attack by ultraviolet light also needs investigations since they are sometimes exposed for long periods of time while in storage.

The permeability of synthetic envelope materials is important to the passage of water from the soil into the drains. Thicker materials will have a measurable permeability but thin materials may be better classified by their porosity. Materials having low permeability have a tendency to perform as filters. Materials of low porosity may restrict water entry and cause sedimentation problems because of their low porosity.

Opening size in the fabric is another factor which needs consideration. If the openings are too large, the soil particles may pass easily through the openings and the drain would not be protected. With random fibre materials it is difficult to determine the effective sizes of the openings. For thinner materials, optical measurements can be made of the distribution and size of openings. For thicker materials, some other technique must be used such as sieving dry materials, with subsequent examination of the particles passing. One method of determining the largest possible opening is by the determination of bubbling pressure.

Soil hydraulic failure

Research currently being conducted at Utah State University in the Department of Agricultural and Irrigation Engineering (Walker, 1978) on the reaction of synthetic drain envelopes to problem drainage soils has resulted in a significant new finding. The failure of the envelope or filter is not related to the characteristics of the envelope material, but rather is a function of the hydraulics of the soil system. The critical hydraulic gradient for structural failure of the soils tested is related to the D60 size of the soil. Structural failure of the soil results in soil particle movement that subsequently clogs the drain envelope material. All synthetic fabrics tested seemed to perform equally well. The studies indicate that the primary function of envelope material is to physically restrain the soil and offer mechanical support. An unsupported soil under conditions of vertical flow will develop a quick condition or unstable condition at a hydraulic gradient of 0.7 to 0.9. By addition of mechanical support in the form of a graded gravel envelope or a synthetic envelope material, the allowable gradient can be increased; or, in other words, the gradients at which soil instability occurs are higher and the soil is less susceptible to degradation.

Subsurface drain functioning

Subsurface drains are normally designed to flow partly full and to function as open channels. However, actually, drains have a limited number of openings and drain spacings are calculated using formulas developed using the assumption of completely permeable drains flowing full with zero back pressure. The discrepancy that exists between these two assumptions results in an error in expected inflow.

Willardson (1967) studied water movement in the soil in the immediate vicinity of a drain. He developed an equation for the equipotentials around a circular drain flowing partly full. He verified his results with a full scale physical model. Inflow to half-full drains was 6 to 16 per cent less than flow into a submerged drain when the drain was completely permeable and 10 to 26 per cent less when the drains simulated drain tiles 30 cm long. The measured loss of discharge due to a drain flowing partially full was attributed primarily to the lost potential difference on the permeable surface of the drain above the water surface.

The effect of cracks and perforations on inflow to drains was evaluated for drains flowing full. Discharge was found to be proportional to the inlet area. Perforations were more efficient than widely spaced cracks. Approximately half of the available driving head in the model was dissipated in the last 1.3 cm of distance radial to a simulated crack opening. At higher flow rates, the 50 per cent energy loss line moved closer to the drain opening. Performance of drains seems to be controlled by hydraulic conditions near the drain. Use of drain envelopes results in closer approximation to the design assumptions made in the development of drain spacing equations.

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Paper 2.03

THE INFLUENCE OF ENVELOPE MATERIALS IN PREVENTING SILTING-UP OF DRAIN PIPES

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Summary

A theoretical analysis has been given about the problem of silting-up of drain tubes through the underside due to flow pressure. The flow pressure depends on the hydraulic gradient, which is determined by the ratio discharge-soil permeability.

A coarse and thick envelope material or a larger drain tube can decrease the danger for silting-up at design norms but cannot prevent it under more severe conditions. To be on the safe side, an envelope material with a higher sand passing resistance be recommended so that the hydraulic gradient may reach at least 2 or 3 times the hydraulic gradient of the design norm.

1. Introduction

Envelope materials in agricultural drainage are mainly used to prevent silting-up of drain tubes, especially in cohesionless soils. They also serve to protect the perforations against blocking and clogging.

Another function of envelope materials is to increase the up-take capacity of drain pipes due to the higher permeability.

To perform these functions, drain tubes were commonly covered with a granular,organic or synthetic material. Nevertheless,it has been ascertained that covering only does not prevent silting-up. Fig.1, 2 and 3 show some of the numerous drain tubes that have been dug up in an experimental drainage field. The soil above drain level was a well structured cohesive soil. The soil under drain level consisted of a cohesionless fine sand (<2 μm : 5%; 2-50 μm : 5%; 50-100 μm : 30%; 100-200 μm : 58%; >200 μm : 2%). That sand was found in the drain tubes. For that reason it is necessary to use an underlayer or complete wrapped drain tubes.

As mostly, drain tube and envelope material are covered with a layer of the well structured top soil, no silting-up is to fear from that side, unless when piping occurs in the unconsolidated trench backfill due to heavy rainfall.

Envelope materials can roughly be arranged into two groups:

- envelope materials with fine pores
- envelope materials with large pores.

It is known that an envelope material with fine pores risks to clog due to fine soil particles, organic dust or iron deposits. Therefore it is recommended to use voluminous envelope materials with large pores.

The phenomenon of silting-up by the underside of the drain and the contribution of an envelope material in preventing silting-up can be explained by means of soil mechanics.

2. Effective and neutral stresses

Consider a vessel with a layer of cohesionless soil (Fig.4) and with the free water level immediately above the soil surface. If the water level is raised to an elevation h_w above soil surface, the normal stress σ_n on a horizontal section increases from almost zero to:

$$\sigma_n = \rho_w \, g \, h_w \tag{1}$$

where

ρ_w = specific mass of water
g = acceleration of gravity.

This increase of the compressive stress does not produce a measurable compression of the soil layer and hence a measurable change of the mechanical properties of the soil as for instance the shearing resistance.

On the other hand by loading the soil surface with a lead shot of the same amount $\rho_q\ g\ h_w$, the compression and hence the mechanical properties of the soil are changed considerably.

Thus the compressive stress in a saturated soil consists of two parts with very different mechanical effects (Terzaghi, 1965). One part, which is equal to the pressure in the water, produces no measurable change of the mechanical properties of the soil. This part is called the neutral stress σ_n and equal to the product of the weight of the water per unit of volume and the height h_w to which the water rises in the piezometric tube at the point of consideration. The piezometric head can be positive or negative and so the neutral stress. The second part produces measurable effects on the mechanical properties of the soil. It is called the effective stress σ_e. The total normal stress σ_t is

$$\sigma_t = \sigma_n + \sigma_e \tag{2}$$

and the effective stress is determined as the difference from the total normal stress and the neutral stress

$$\sigma_e = \sigma_t - \sigma_n \tag{3}$$

For example, consider a vessel with a saturated soil (Fig.5) with a specific mass ρ_s. The water level reaches a height H above the soil surface and the piezometric h_w at a depth z below the soil surface is

$$h_w = H + z \tag{4}$$

The neutral stress is

$$\sigma_n = \rho_w\ g\ (H + z) \tag{5}$$

and the total normal stress

$$\sigma_t = \rho_w\ g\ H + \rho_s\ g\ z \tag{6}$$

Hence, the effective stress at a depth z below the soil surface is

$$\sigma_e = \sigma_t - \sigma_n = (\rho_s - \rho_w)\ g\ z \tag{7}$$

As $(\rho_s - \rho_w)$ presents the submerged specific mass ρ', than from the preceding equation 7 is obtained

$$\sigma_e = \rho' \ g \ z \tag{8}$$

3. Flow pressure and hydraulic gradient

We just considered water which is in static equilibrium and which occupies the voids.

Consider now the system in which the water percolates through the voids (Terzaghi and Peck, 1965).

A vessel A contains a sand layer supported by a metallic screen (Fig.6). The sand layer has a thickness H and the water level reaches a height H_1 above the surface of the sand layer. The space under the metallic screen communicates with a vessel B by means of a tube. The water level in each vessel is the same. As long as the water in the two vessels is maintained at the same level, the effective stress σ_e at a depth z below the surface of the sand layer is

$$\sigma_e = (\rho_s - \rho_w) \ g \ z \tag{9}$$

When the vessel B is lowered over a height h, water flows through the sand layer in downward direction with a hydraulic gradient

$$i = h/H \tag{10}$$

The neutral stress at a depth H below the surface of the sand layer reduces with a value

$$\rho_w \ g \ h = i \ \rho_w \ g \ H \tag{11}$$

At a depth z below the surface of the sand layer, it takes a value of

$$\Delta \sigma_n = i \ \rho_w \ g \ z \tag{12}$$

and the effective stress increases with the same amount.

When the vessel B is raised over a height h, the water flows in upward direction and the neutral stress at a depth z below the soil surface increases with

$$\Delta \sigma_n = i \rho_w g z \tag{13}$$

The effective stress decreases so that

$$\sigma_e = \rho_s g z - \rho_w g z - i g \rho_w z = (\rho_s - \rho_w - i \rho_w) g z \tag{14}$$

The increase $\Delta \sigma_n$ of the neutral stress is due to the fact that the pore-water passes from static equilibrium into dynamic equilibrium. The corresponding change $\Delta \sigma_n$ of the effective stress in the sand is called flow pressure. Thus the flow pressure is proportional to the hydraulic gradient.

$$\Delta \sigma_n = i \rho_w g z \tag{15}$$

It is the result of the friction between the flow of water and the walls of the voids. If the water flows downwards, the flow solicits the sand particles downwards and increases, as a consequence, the effective stress in the sand. On the contrary, by an upward flow, the friction between the flow of water and the walls of the voids tends to lift-up the sand particles. As soon as the hydraulic gradient i equals

$$i_c = \frac{\rho_c - \rho_w}{\rho_w} \tag{16}$$

the effective stress becomes zero in every point of the sand layer and the critical hydraulic gradient i_c has been reached as well as the situation of quick-sand.

In this state, erosion will occur if no precautions are taken.

4. Vertical flow problem

Applying an envelope material with a certain thickness and permeability above a sand layer, through which upward flow takes place, gives a flow rate:

$$Q = k_s J_s A = k_f J_f A \tag{17}$$

where

k_s = permeability of the soil
k_f = permeability of the envelope material
J_s = hydraulic gradient in the soil
J_f' = hydraulic gradient in the envelope material
A = cross section area

From 17, it follows that

$$J_f = \frac{k_s}{k_f} J_s \tag{18}$$

If the critical hydraulic gradient in the soil is reached and the permeability of the envelope is 100 times the permeability of the soil, we obtain

$$J_f = \frac{1}{100} J_c < J_c \tag{19}$$

and even when the hydraulic gradient is higher than the critical value, J_f remains smaller than J_c, and theoretically sand particles accumulate in the envelope material.

Supposing a very coarse envelope material without resistance for sand passage. If the critical hydraulic gradient has been reached, when no envelope material is used, it is clear from Fig.7 that applying the same hydraulic head in using an envelope material leads to a higher hydraulic gradient in the sand and hence silting up of the envelope material occurs.

For the same discharge, the same hydraulic gradient in the sand has to be applied using an envelope material (Fig.8), although the total head loss shall decrease depending on the thickness of the envelope material.

When a coarse envelope material is used and the critical hydraulic gradient has been reached in the sand, the sand is hoving in the envelope by increasing hydraulic gradient till it reaches the flattened drain tube. Another phenomenon must be taken into consideration. At the moment when J_c is reached, the binding forces between the particles disappear and the permeability of the sand increases considerably, as well as the discharge.

If the envelope material has a certain resistance against sand passage, soil particles are retained and pushed into the envelope by increasing the hydraulic gradient. The flow pressure presses the whole sand column against the envelope and the soil particles do not lose their binding forces. In this way not the whole envelope is saturated with sand particles but some large voids make transport of sand particles towards the drain possible.

The possibility in applying a higher hydraulic gradient depends on the resistance of the envelope material against sand passage as shown in Table 1, which gives the hydraulic gradients and discharges at the moment that sand particles were passed through the envelope and deposited on the flattened drain pipe.

TABLE 1. Hydraulic gradients and discharges for different types of envelope materials at the moment that sand was deposited on the flattened drain pipe

Envelope material	Mean hydraulic gradient in the soil J_s	Discharge $Q(cm^3/sec)$
no envelope	1.22	1.79
envelope with very slight resistance to sand passage	2.17	7.83
envelope with slight resistance to sand passage	2.47	5.09
envelope with high resistance to sand passage	5.53	11.24
envelope with still a higher resistance to sand passage	6.55	10.60
envelope with a high resistance to sand passage	6.74	11.38

5. Radial flow towards a drain tube

Considering a radial flow towards a drain tube, the potential loss is:

$$\Delta h = h - h_o = \frac{q}{2\,\pi\,k_s} \ln \frac{R}{R_o}$$

where

h = hydraulic head at a distance R

h_o = hydraulic head at a distance R_o

R = radius of an equipotential

R_o = drain radius

q = discharge per unit drain length

k_s = permeability of the soil

and the hydraulic gradient

$$J = \frac{\Delta h}{\Delta R} = \frac{q}{2\,\pi\,k_s}\ \frac{1}{R}$$

Taking a drain tube with a diameter of 5 cm and a constant ratio q/k_s = 0.182 m that can occur in practice (drain distance 12 m; k_s = 0.46 m/day; N = 7 mm/day) the hydraulic gradient is the highest near the drain tube (Fig.9) and hence the danger for erosion (van der Beken, 1968).

Using an envelope with a thickness of 2 cm and a permeability which is 100 times the permeability of the surrounding soil; the hydraulic gradient at the drain tube is diminished 100 times, but as we have seen this does not give guarantee against silting-up when a coarse envelope has been used. But also the hydraulic gradient at the interface soil-envelope is smaller and silting-up of the envelope material is prevented (Fig.10) under the given circumstances.

For the same discharge, the piezometric head is only 4.8 cm with envelope material instead of 6.5 cm without envelope. If the piezometric head is kept on 4.8 cm at a distance of 10 cm, it can be calculated that the discharge of the drain without envelope gives only 58% of the discharge of the drain with envelope.

The same effect can be obtained using a larger drain diameter.

If the ratio q/k_s is doubled, the danger for silting-up of drain tubes without envelope increases (Fig. 9) and also with envelope, at the interface soil-envelope, silting-up of the envelope and the drain tube has become possible (Fig. 10) depending on the resistance to sand passage.

6. Conclusions

It has been ascertained from field experiments that silting-up of drain pipes can occur through the underside of the drain tube, especially in cohesionless soils. It has been proved theoretically that sand can invade the drain pipe through the underside due to the flow pressure, which depends on the hydraulic gradient. The hydraulic gradient is determined by the ratio q/k_s. At design norms silting-up easily can occur depending on the permeability of the soil.

An envelope material with a higher permeability than the surrounding soil has a favourable influence on the hydraulic head. To find such an envelope material is not a problem but to find an envelope material that does not clog due to fine soil particles, organic dust or iron deposits is more complex. An envelope material with fine pores is therefore only applicable in soils with a small content of clay (< 2 µm) and loam (2-50 µ) particles without organic matter and no risk for iron deposits. If this is not the case, a voluminous envelope material with coarser pores is to be recommended.

Using a coarse envelope material can diminish the risk of silting-up, as well as a larger drain diameter. To have more security against silting-up it is preferable to apply an envelope material with a higher resistance to sand passage so that the hydraulic gradient may reach 2 or 3 times the hydraulic gradient of the design norms in order to be safe.

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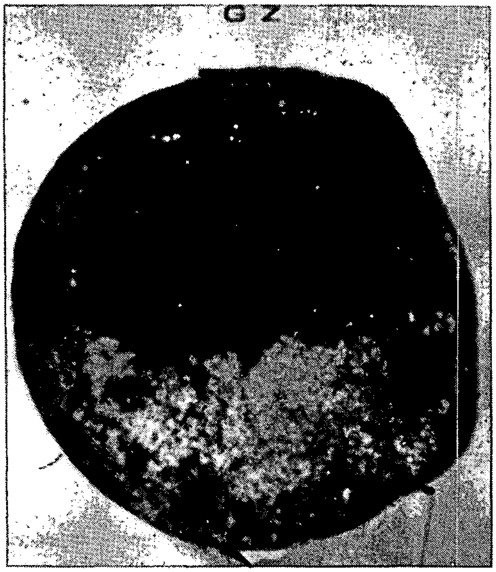

Fig.1. Silting-up of a clay pipe. No envelope material was used. Note the iron deposit on the sand layer (dark part).

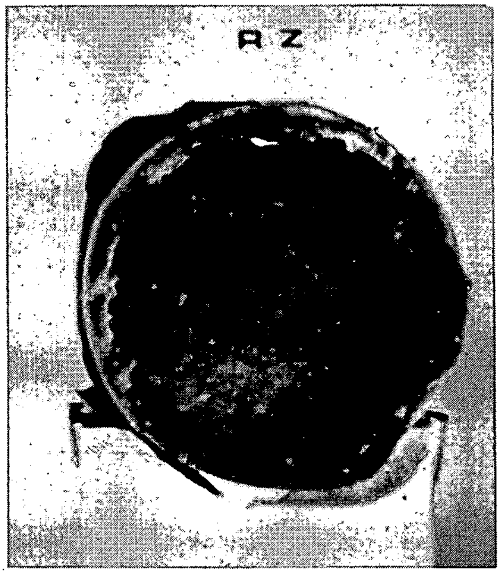

Fig.2. Invasion of sand into a corru-plastic pipe without envelope mate-rial.

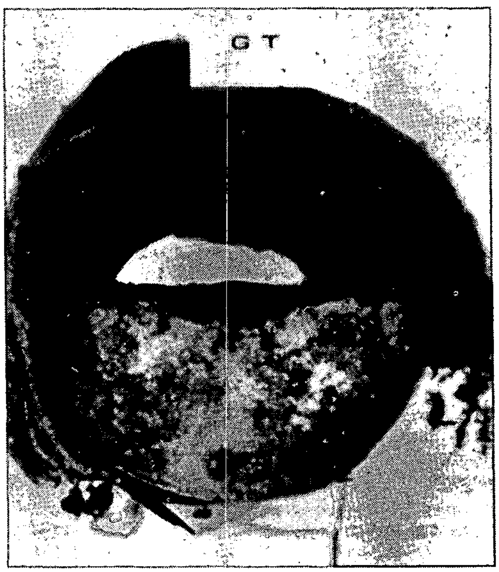

Fig.3. Silting-up of a clay pipe covered with a peat layer.

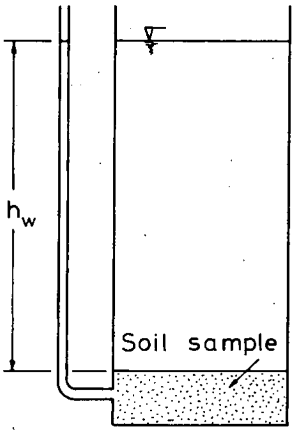

Fig. 4 Illustration of the normal stress on an horizontal section in a sand layer

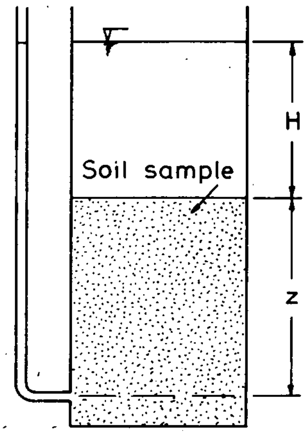

Fig. 5 Illustration of the total normal stress on an horizontal section in a sand layer

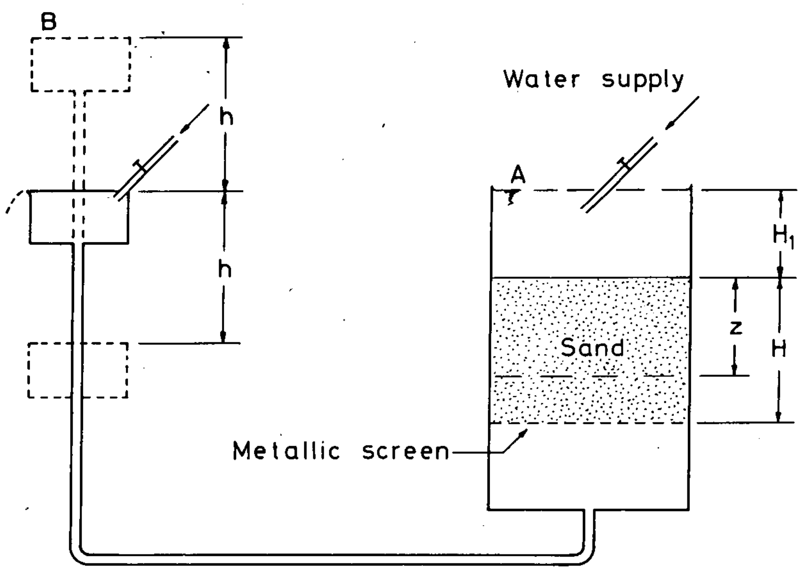

Fig. 6 Illustration of the influence of the flow pressure

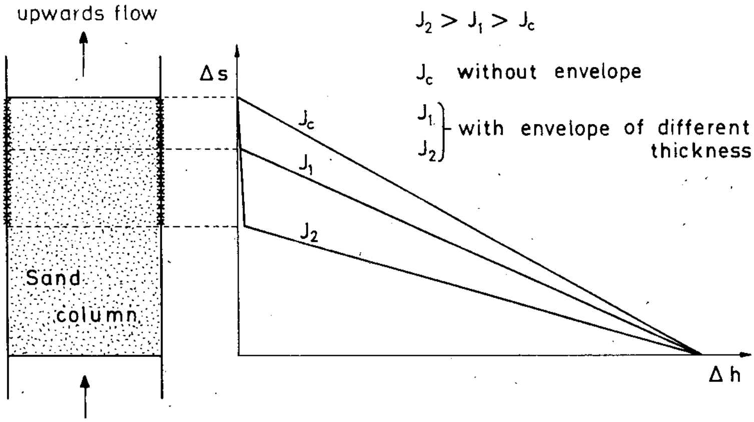

Fig. 7 Hydraulic gradients for the same hydraulic head

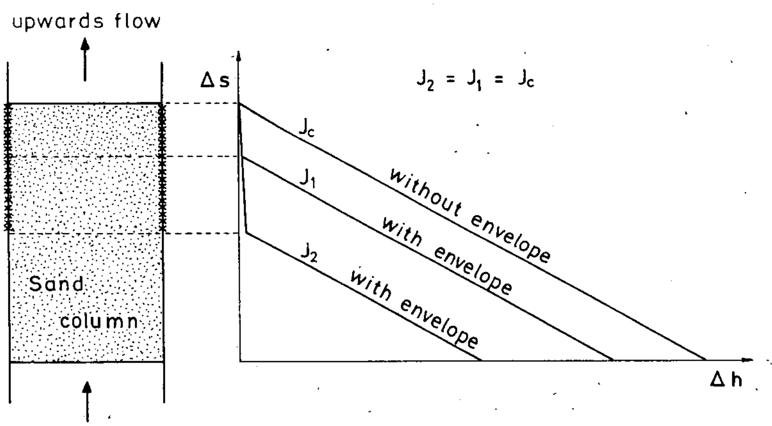

Fig. 8 Hydraulic gradients for the same discharge

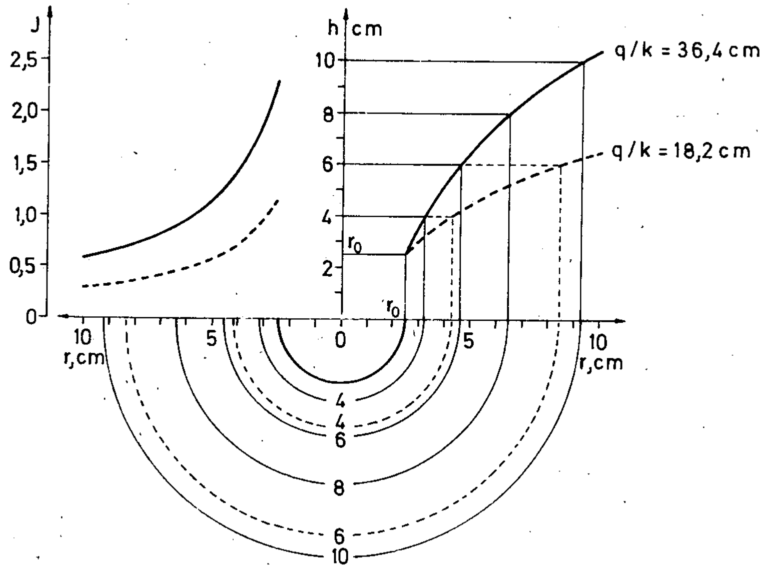

Fig. 9 The piezometric head and hydraulic gradient near an ideal drain tube

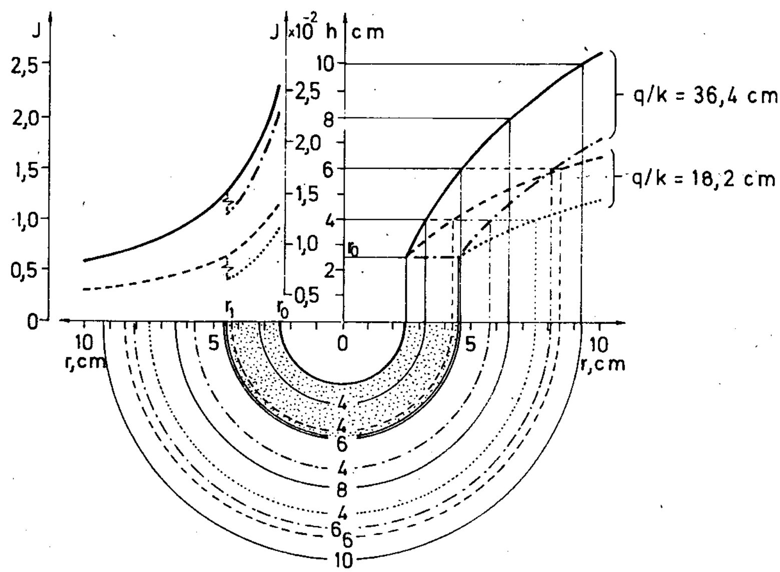

Fig. 10 The piezometric head and hydraulic gradient near an ideal drain tube with an envelope material

Paper 2.04

STABILIZED SOIL REPLACING ENVELOPE MATERIALS

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Summary

A laboratory investigation has been carried out on the possible use of soil conditioners in stabilizing unstable sandy soils in order to prevent silting-up of drain pipes or to increase the permeability of the trench backfill for low permeable soils. Different kinds of soil conditioners are investigated in function of the drying time for different types of soils. Although the investigations are not yet finished, some positive results can be attributed to the soil conditioners under consideration.

1. Introduction

The most common type of pipe drainage installation consists of excavating the trench, installing the pipes in it and then filling it back with the excavated soil. Experiences have shown that there are three possible causes of a low drainage efficiency (Dierickx and Gabriels, 1976):

- blocking and stoppage of drain tubes due to the silting-up phenomenon which takes place in unstable soils;

- the low permeability in the trench near the drain tube caused by structure deterioration when drainage has been carried out in unfavourable conditions;

- the low permeability in the trench of heavy badly structured loam and clay soils.

If conditions for silting-up are favourable, drain tubes are usually covered or wrapped with protective materials such as peat, flax straw or coconut fibre, but in many cases blockage and stoppage of the envelope materials can still occur.

Until now the low permeability of the trench refilled with the original heavy soil remains a serious problem, while refilling the trench with gravel is an expensive enterprise.

In each of these cases, it is of great importance to ensure a good and stable structure in the vicinity of the drain tube and especially in the whole trench for low permeable soils, because the greatest amount of the water to be drained from the surface has to pass through it.

It is known since a long time that the structure of a soil can be modified by adding suitable improvers at extremely low rates. Polymer solutions and polymer emulsions have been developed for that purpose. It should be attractive to make stable aggregates out of the local soil by applying soil conditioners and using them as backfill. In such a way the success of a drainage system in unstable soils and even in very impervious soils should be assured.

2. The behaviour of soil conditioners

The used soil conditioners can be divided into two groups: polymer solutions and polymer emulsions. A different mechanism is observed when soluble and emulsified adhesives are compared (van de Velde and de Boodt, 1972).

All polymer solutions appear to consist of a complex fibrous material having a treadlike structure holding the soil particles together by strands of soil conditioner material.

In the case of emulsions, it is supposed that electrically charged micelles glide over the thin water films, surrounding the mineral particles, into the menisci at the contact points between the particles when the soil is drying out. In this last case the particles stick together of flocculated circular micelles in the dried-out menisci. Previous wet sieving experiments have shown the existence of a relationship between the total amount of liquid (initial soil moisture content + soil conditioner + dilution rate) added to the soil and the aggregate stability. It has been established that the optimal moisture content for sand is about 20 per cent and for loamy and clayey soils about 25 per cent on dry weight basis (Gabriels and de Boodt, 1972; Rigole and de Bisschop, 1972). It has also been observed that the stabilized soil needed drying during 24 hours at about 20 per cent. This latter creates of course a great problem in practice, especially under humid conditions.

In view of this new possibility with soil conditioners, laboratory investigations were carried out to evaluate the effect of the present-day soil conditioners on the structure stability of the treated soils used as backfill. Also the influence of the concentration of the soil conditioners and the drying time after treatment were investigated.

3. Laboratory research

2.1 Treatment of the soil

The used soils were a dune sand, a sand loam and a heavy clay (Table 1).

TABLE 1: Particle size distribution and organic matter content of the used soils

Fraction (μm)	sand	sand loam	heavy clay
0-2	0.0 %	19.0 %	50.5 %
2-50	0.4 %	58.3 %	47.7 %
> 50	99.6 %	22.7 %	1.8 %
% organic matter	0.08%	0.31%	0.76%

Before treatment, the loamy and clayey soils were passed through a sieve with mesh size of 3 mm. The three kinds of air-dried textures were treated with six products at different concentrations (Table 2). The dilution was such that the moisture content of the soils after treatment was about 20 per cent. Spraying and mixing the soil were done simultaneously and the aggregates thus formed were allowed to dry completely, 24 hours and 2 hours at room temperature. After the drying cycle, the loam and clay samples were passed through three sieves with mesh sizes of 2, 3.36 and 4.76 mm. A new sample was made with these three fractions as follows:

- 20 per cent of aggregates less than 2 mm;
- 40 per cent of aggregates between 2 and 3.36 mm;
- 40 per cent of aggregates between 3.36 and 4.76 mm.

In this way the difference brought about in soil aggregation was eliminated.

TABLE 2. The used soil conditioners

Form	Trade name	Basic product	Concentration*
solution	Humofina PAM	Polyacryl amide	$2^{o}/oo + (1^{o}/oo)^{\bullet}$
	Uresol 310	Polyurethane	$6^{o}/oo + (3^{o}/oo)^{\bullet}$
	Lima 1110	Ligmosulphonate	$5^{o}/oo + (2.5;1;0.5^{o}/oo)^{\bullet}$
emulsion	Humofina Bitumen	Asphalt	1%
	Petroset DB	Butadiene-styrene	1%
	Curasol AE	polyvinyl acetate	1.5%

* The percentage ratio of active material on dry soil weight

• Only with sand loam after 2 hours of drying

3.2 Permeability test

To judge the validity of the soil conditioners and to get an idea about the breakdown of the formed aggregates by the running water, the permeability of the treated samples was measured in function of time. A cylinder was filled with a sample and water was flowing through the soil in a stationary situation during 19 days. The soil sample has a height of 22 cm and a diameter of 10.2 cm. The soil was kept in the cylinder by a coarse screen with a mesh size of 0.1 cm, fixed at 3 cm above the bottom of the cylinder. Figure 1 shows the experimental set-up schematically. The permeability at a certain temperature can be calculated by DARCY's law.

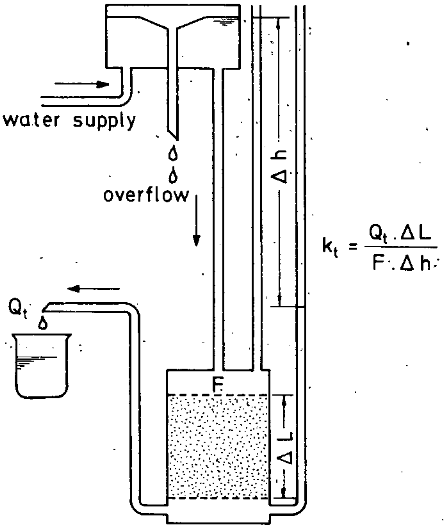

Fig. 1 Experimental set-up for measuring the permeability

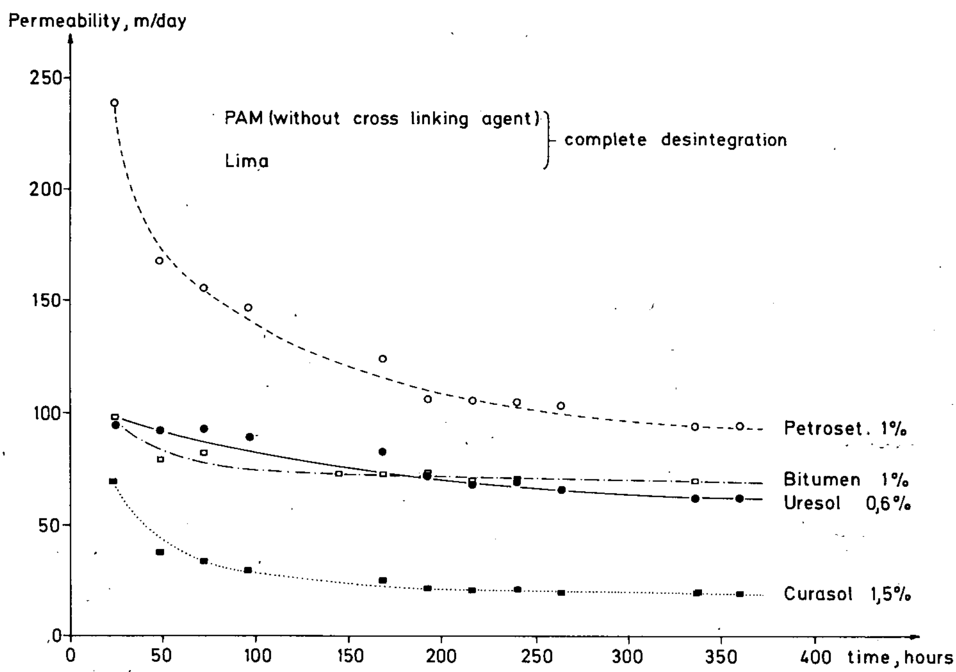

Fig. 2 , Permeability of air-dried stabilized sand in function of time

4. Results

4.1 Sandy soils

Sandy soils do not have permeability problems compared with loam and heavy clay soils. The purpose of measuring the permeability of the treated sandy soils is only to evaluate the effect of different soil conditioners on the resistance against breakdown of the sand aggregates due to the running forces of the water.

Decreasing permeability with time indicates a structural breakdown and in consequence the inefficiency of the used conditioner. Figure 2 gives the permeability values as a function of the duration of the experiment for a cohesionless sandy soil completely air-dried after treatment. From the results it can be seen that the permeability decreases in the beginning and becomes constant after about 6 days. The decrease in permeability can have different causes:

- The water makes the binding agents of soil conditioners weaker, more flexible or elastic, resulting in a more dense packing of the aggregates, especially at the bottom of the sample, or in a breakdown of the aggregates into smaller ones.
- When the granulates of the dry soil are immersed in water, all the outer pores are filled with water and surface tension pulls water into many of the coarser pores. This compresses the air in the granulates, which can only escape by bursting the granulates into smaller ones. This disruption will be greater when the wetting velocity is faster (Cernuda et al., 1954).
- Running water exerts shear forces on the surface of the crumbs which in turn slake into fine granules. Fine granules, slaked from aggregates, fill up the coarser pores in the samples and the resulted pores may be firmly interlocked. Water flow through soil pores is limited by such numerous constrictions and dead-end spaces.

It must be said that no soil particles or native material left the samples during the experiment. That agrees with previous experiments (Bishay et al., 1975). The experiments have shown that, using sandy soil conditioned with bitumen (0.75 per cent) and PAM (0.2 per cent) as an artificial envelope around the drain tube, silting-up was prevented and that the discharge was 1.3 times bigger than the discharge from a drain without envelope. In the experiment with PAM a cross linking agent (glyoxal) was used. Trials without that cross linking agent did not give any effect. Indeed the sandy samples conditioned with PAM leached through the coarse screen,as well as sandy samples treated with Lima. Soils with a low content of clay and/or humus need a cross linking agent when treated with PAM (van de Velde and de Boodt, 1972). Probably also sandy soil treated with Lima need a cross linking agent. Figure 2 shows that the permeability of the treated sandy soil (especially with Petroset, Bitumen and Uresol) is higher than the one of the untreated soil, which has a permeability of about 17 m/day.

Highly permeable sand surrounding a small plastic tube reduces the head losses in the vicinity of the drain tube and facilitates, and increases the water flow towards the drain. In cohesionless sand these last properties are in fact positive but adventitious compared with the prevention of silting-up. The results of the permeability tests on treated sand dried during 24 hours after treatment are given in Table 3.

TABLE 3. Permeability values (m/day) of stabilized sand dried during 24 hours

Products	Measurement time		
	3 days	10 days	19 days
Bitumen	complete disintegration		
PAM (without glyoxal)	complete disintegration		
Lima	complete disintegration		
Uresol	experiments not yet carried out		
Curasol	experiments not yet carried out		
Petroset	127	112	78

It is obvious that stabilized sand needs a long drying time before the formed aggregates can resist breakdown when put in contact with water. Only Petroset gives a positive result after a drying time of 24 hours.

4.2 Sand loam soil

The low efficiency of a drainage system in unstable loam soils and heavy soils can be attributed to the low permeability of the trench. Many investigations have proved that the trench has an important function in low permeable soils in order to obtain a satisfying drainage system. The effect in treating these soils with soil conditioners on the permeability has been investigated. Table 4 gives a review of the permeability values of sand loam as a function of duration, drying time and used product.

When the soil was air-dried after treatment, it must be stated from Table 4 that the permeability of the treated soil is very high, 10 to 200 times that of the untreated one. Besides, it was not possible to measure th permeability of the untreated soil in the same way as for the stabilized soil. A filter material was needed to prevent leaching of the sample throug the screen.

During the first days of the experiments the permeability moderately decreased and then took a nearly constant value. The same phenomenon was already established in the sand experiment and consequently the same causes are valid here.

The permeability of soil treated with Lima and Uresol decreased continously in function of time. Perhaps the disintegrating forces caused by a greater water velocity in accordance with a higher permeability may lie on the basis of this phenomenon.

It may be remarked that the soil stabilized with emulsions gives lower permeability values than the soil treated with solutions. That difference will be more pronounced when the drying time becomes shorter.

After a drying time of 24 hours, the samples had a moisture content of about 11 per cent. It is conspicuous that the treatments with emulsions give much lower permeability values than the ones obtained after air-drying They even approach the permeability values of the untreated soil. The inefficiency of the emulsions can be attributed to the fact that the water content of the soil is still too high before the micelles coagulate together in the contact points between soil particles and consequently leach out of the soil samples. The permeability values of the soil treated with solutions are greater than the ones obtained with the air-dried soil. It is generally known that wetting a soil with higher moisture content gives a smaller break-up of granulates.

TABLE 4. The permeability values (m/day) of the treated sand loam

Drying time	Product		Concentr.%	3 days	10 days	19 days
Air-dried	Solution	Lima	0.5	1719	1393	882
		Uresol	0.6	1245	342	701
		PAM	0.2	569	493	463
	Emulsion	Curasol	1.5	184	156	131
		Bitumen	1	161	117	83
		Petroset	1	70	54	45
	Untreated			3.6	3.0	2.4
24 hours	Solution	PAM	0.2	1427	1226	1149
		Lima	0.5	1347	1141	1036
		Uresol	0.6	2626	1058	807
	Emulsion	Bitumen	1	24	27	24
		Curasol	1.5	48	30	28
		Petroset	1	9	8	6
	Untreated			2.9	2.7	2.3
2 hours	Solution	Uresol	0.6	1342	1041	755
			0.3	276	213	212
		PAM	0.2	1092	922	663
			0.1	782	670	541
		Lima	0.5	417	325	247
			0.25	381	338	241
			0.10	132	147	82
			0.05	80	41	25
	Emulsion	Petroset	1	20	8	4
		Curasol	1.5	59	39	4
		Bitumen	1	complete disintegration		
	Untreated			3.9	2.7	2.1

After a drying time of two hours, the samples had a moisture content of about 17.5 per cent. As could be expected the treatments with emulsions are ineffective. The stability effected by polymer solutions remains remarkably high.

Applying solutions at half concentrations leads to about the same permeability values, except for Uresol, as can be seen from Table 4. A further lowering of the concentration results in a decrease of the permeability.

4.3 Clay soils

The experiments with the stabilized clay soil are not yet finished but the preliminary results, as given in Table 5, allow to conclude that soil conditioners will have a favourable influence on the stability of heavy clay soils. The permeability values of the stabilized clay soils are 10 to 100 times greater than the values of the untreated soils. Of course, the values are smaller than of loamy soil, as the swelling effect which is extremely high in this clay soil, causes a remarkable decrease of the permeability.

5. Economical aspects

The practical application of soil stabilizers will largely depend on the costs. At present only prices of the products are available while practical experiments will permit to calculate the implementation costs.

Table 6 gives the cost price of the used soil conditioners compared with the price of gravel and coarse sand. As can be deduced from the table, only Bitumen, PAM and Lima (at the lowest concentration) compete with gravel and coarse sand. Taking into account the high transport cost of gravel and sand, soil conditioners seem attractive from an economical view-point. Soil conditioners will certainly be economical in replacing graded gravel envelopes.

6. Conclusions

From the obtained laboratory results it is obvious that most products have a favourable influence on the aggregate stability when the treated soil is completely air-dried. Under humid conditions the solutions of soil conditioners, which are unsoluble in the soil and which are easy and clean to handle, seem to be very suitable. Soil conditioners can prevent silting-up of unstable soils and improve the permeability of the drain trench of less permeable soils as loamy and clayey soils.

There are however still many questions to solve as:

TABLE 5. The permeability values (m/day) of the treated clay soil

Drying time	Product		Concentr.%	3 days	10 days	19 days
Air-dried	Solution	Uresol	0.6	175	100	87
		PAM	0.2	77	51	32
		Lima	0.5	49	38	13
	Emulsion	Petroset	1	244	162	108
		Bitumen	1	7	3	1.5
		Curasol	1.5	1.5	0.5	0.4
	Untreated			0.30	0.45	0.40
24 hours	Solution	Uresol	0.6	318	235	140
		Lima	0.5	147	90	70
		PAM	0.2	133	93	50
	Emulsion	Bitumen	1	139	96	80
		Petroset	1	121	89	55
		Curasol	1.5	28	18	13
	Untreated			1.15	1.11	0.82

TABLE 6. The costprice of soil conditioners, gravel and coarse sand

Product	Price of product (in BF) •	Prices in BF per metre of drain length	
		cover layer of 5 cm height	whole trench 80 cm depth
PAM	30/liter	5,25 (0,2 %) • • 2,62 (0,1 %)	84 (0,2 %) 41,92 (0,1 %)
Uresol	120/liter	13,10 (0,6 %) 6,55 (0,3 %)	209,66 (0,6 %) 104,83 (0,3 %)
Lima	58,5/liter	20,48 (0,5 %) 10,24 (0,25 %) 2,05 (0,05 %)	327,68 (0,5 %) 163,84 (0,25 %) 32,77 (0,05 %)
Bitumen	12/liter	3,36 (1 %)	53,76 (1 %)
Petroset	60/liter	16,80 (1 %)	268,80 (1 %)
Curasol	60/liter	25,20 (1,5 %)	403,20 (1,5 %)
Gravel • • •			
10 - 63	200/ton	2,80	44,80
8 - 22	230/ton	3,22	51,52
Coarse sand	230/ton	3,45	55,20

• *32,6 BF = 1 U.S. $ (June 1978)*

• • *concentration of active material for dry soil*

• • • *gravel gradation (mm)*

- the loading effect on the stability and the permeability of the stabilized soil;
- the influence of salt on the stability of the treated soil;
- the lasting effect of the soil conditioners;
- a suitable and economical method for stabilizing the backfill in practice.

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Paper 2.05

ON THE EFFICIENCY OF FILTERS IN DRAIN TRENCHES

(Since this paper has been published in english as Information Bulletin 11 of the Bundesanstalt für Kulturtechnik und Bodenwasserhaushalt, Petzenkirchen, Austria, only the abstract will be included in these Proceedings.)

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Abstract

With regard to the economic use of financial resources the most important consequence of the present investigation results was the suitability of cohesive soils for the refilling of drain trenches without using special filter materials. A simple qualification test is described as an aid for decisions on the sufficiency of cohesion.

The well-known filter-rules, which were tested in many other fields of hydraulic engineering, proved their full validity for drainage problems. Design criteria for some special filter problems need an deepened acquisition, the fundamental understandings for their solution are available.

The working mechanism of filter materials was described by two models for the most frequent local conditions. For many practical cases the conclusions are permitting a proper decision for the choice of filter material and for the technical solution of the problem.

An order of filter materials and techniques by their efficiency under certain hydropedological conditions or by the economy of their application is not to be given in a generally accepted way. The solution of the special filter problem must be found for the present local conditions by optimation of all aspects of technical and economical relevance.

Paper 2.06

TESTING SYNTHETIC FABRICS FOR USE WITH DRAINAGE CONDUITS

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Summary

Due to the desire for high-speed installation of subsurface drains, there has been a great emphasis on the development of an effective synthetic fabric envelope material. A task force has been established by ASTM to develop test methods for evaluating these materials. The task force is evaluating a test to determine the amount of soil piped through a filter as well as a test to measure the average pore size and equivalent opening size of fabric filters. Other tests to determine the chemical and mechanical properties are being studied by the task force. Evaluation of some United States subsurface drains protected with fabric filters has allowed us to have confidence in these materials if the silt fraction (0.05 mm to 0.002 mm) in the soil around a conduit is less than 40 percent.

Introduction

Many soils in the world have a particle-size distribution that creates serious sedimentation problems when subsurface drains are installed. Fine sands and silts are two of the major problem soils. Over the years efforts have been made to prevent soil particles that will not stay in suspension from entering the drain. The most effective protection has been provided by sand and gravel envelopes. These are designed to improve flow characteristics around the drainage conduit and to restrain larger particles from flowing into it.

Many other materials have been used as the envelope, such as wood chips, cocoa fiber, straw, organic materials, and industrial slag. Some have been quite effective, but a few are compressible and allow excessive deflection when used with flexible conduits such as corrugated plastic tubing. In recent years, synthetic fabrics have been developed and marketed in an effort to solve this sedimentation problem with a low-cost material that can be pre-wrapped around drainage conduits.

American Society for Testing and Materials (ASTM)

Because of a desire for high-speed installation, there has been great emphasis on the development of an effective synthetic fabric envelope material, sometimes called a filter. In October 1976, the American Society for Testing and Materials (ASTM) organized a task force under the Land Drainage Systems Subcommittee of its Plastic Piping Committee. The task force is developing test methods for evaluating filter fabrics for subsurface drainage systems.

The members of the task force evaluated many industry and government tests used for examining various fabric products. They decided to initiate round-robin evaluations for two of the tests they thought looked promising. (Round-robin evaluations are tests performed by different individuals using the same procedures, similar test equipment, and identical samples.) One of the fabric tests uses a 10-inch (254-mm) diameter permeameter to determine the amount of soil piped through a filter under a constant head of water. The other test measures the average pore size and equivalent opening size of fabric filters through standard sieving techniques using closely graded spherical glass beads.

The ASTM task force is now conducting the round-robin tests. The members are running the tests in six different locations with nearly identical samples of two soil series. In this way they hope to determine whether the tests are reproducible and whether they need to be modified. The soil samples were collected, prepared, and distributed by the USDA Soil Conservation Service (SCS).

The ASTM task force realizes that other tests, besides the two mentioned, will be needed to determine the chemical and mechanical properties necessary for a satisfactory drainage envelope material. Therefore, the members are studying the usefulness of existing textile tests, to be modified as necessary, for:

1. Tensile strength.
2. Bursting strength.
3. Puncture strength.
4. Abrasion resistance.
5. Chemical resistance.
6. Ultraviolet resistance.
7. Tear strength.

USDA Soil Conservation Service (SCS)

SCS evaluated 33 soils from six midwestern states that generally require a filter to prevent sediment from entering the drain. After studying detailed laboratory analyses of these 33 soils, we recommended six that would best represent the range of problem soils. The ASTM task force reviewed our recommendations and selected two - the Sanilac and Bruce soil series - for the round-robin testing.

SCS obtained a representative sample of the Sanilac series in Huron County, Michigan. The sampling depth was 13 to 52 inches (330 to 1,320 mm). The plasticity index was 4, and fines (particles less than 0.05 mm in size) make up 92 to 94 per cent of the sample.

A representative sample of the Bruce series was obtained in Chippewa County, Michigan. The sampling depth was 7 to 22 inches (178 to 559 mm). The plasticity index ranges from 6 to 8, and fines make up 43 to 46 per cent.

The SCS Soil Mechanics Laboratory obtained large samples from these sites. The sample from each site was first mixed by tumbling it in metal drums that contain rubber-covered steel rods. Then it was placed in 12 large containers. To ensure that the total sample of each series was uniform, the containers were lined up in three tiers and the four samples in each tier blended together. Then one sample of each tier was blended with one sample of each of the other two tiers. The resulting three blends were labelled A,B,and C, and gradation and Atterberg limit tests were made on each. The data from these tests (Table 1) indicated that the blending produced nearly identical samples of each series. The SCS Mechanics Laboratory then distributed a 25-lb (11.3-kg) sample of each soil series to each of the round-robin testers.

In addition to the two samples sent for round-robin testing, SCS has collected 200-lb (91-kg) samples from three other soil series for use in other

TABLE 1. Test data results on soil sample blends

Soil series and sample blends	Grain size distribution (expressed as per cent finer by dry weight)									Atterberg limits		Unified classification
	0.002	0.005	0.02	0.05	0.074	0.105	0.250	0.42	0.84	liquid limit	plasticity index	
Bruce series												
Blend A	11	21	28	43	56	73	99	100	-	23	8	CL
Blend B	11	21	28	44	54	71	98	99	100	23	8	CL
Blend C	12	22	28	46	56	71	98	99	100	23	6	CL-ML
Sanilac series												
Blend A	9	17	47	94	100	-	-	-	-	22	4	CL-ML or ML
Blend B	9	17	47	93	100	-	-	-	-	22	4	CL-ML or ML
Blend C	9	17	47	92	100	-	-	-	-	22	4	CL-ML or ML

evaluations: the Wakeland, Huey, and Gilford series. SCS will use these soil to develop general standards for fabric filters. When the information from these soil series is added to that from the Bruce and Sanilac series, we hop to have enough data for trial standards.

A representative sample of the Wakeland series was obtained in Spencer Count Indiana. Its sampling depth was 8 to 60 inches (203 to 1,524 mm). Its plasticity index is 4 to 10, and fines make up 10 to 20 per cent.

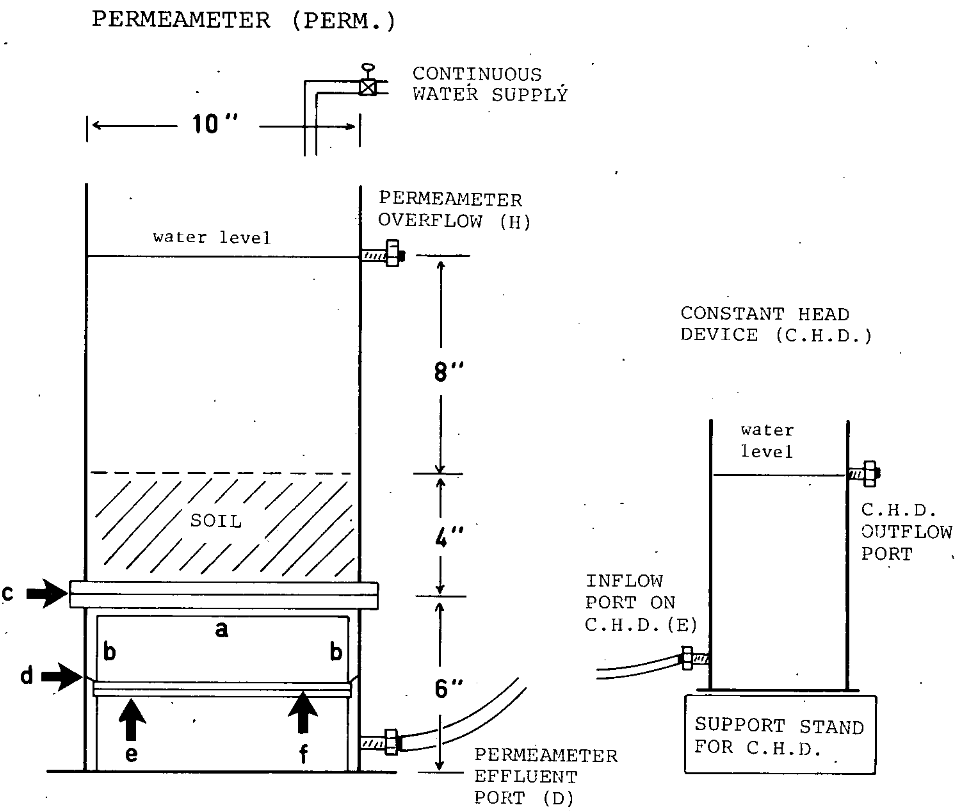
SCS obtained a representative sample of the Huey series in Jasper County, Indiana. The sampling depth was 27 to 57 inches (686 to 1,448 mm). Its plasticity index is 28 to 29, and fines make up 91 per cent. A pore water chemistry analyses indicated 15.8 meq/l salts in the saturation extract and 97.8 per cent of this is sodium thus it is a dispersed soil sample.

A representative sample of the Gilford series was obtained in Pulaski County Indiana. The sampling depth was 12 to 34 inches (305 to 864 mm). Its plasticity index is 7 to 8, and fines make up 33 to 34 per cent.

Test procedures

The ASTM task force is using a typical permeameter with a constant head device to perform the soil permeability tests (Fig.1). An 8-inch (203-mm) head of water is maintained over a 4-inch (102-mm) depth of soil in a 10-inch (254-mm) diameter permeameter. The filter fabric to be tested must be 10-1/2 inches (267 mm) in diameter. It is placed under the soil sample, and mounted below it is a microporous fabric media. Four piezometers are installed in the test apparatus. Three are placed at points 1/2, 1-1/2, and 3 inches (13, 38, and 76 mm) above the filter fabric. The fourth is placed in the chamber between the microporous filter and the filter fabric.

The other test procedure being checked by the ASTM task force uses dry sieving to determine average pore size and equivalent opening size of filter fabrics. It is an adaptation of a procedure using standard glass beads that was developed by the US Army Corps of Engineers (1977). A bead range distribution curve is developed, and standard procedures for sieve analysis are used. In brief, the fabric is attached to a standard sieve having openings larger



(a) filter fabric support frame (B)
(b) threaded rods that support microporous filter mounting bracket (A) and filter fabric support frame (B)
(c) subgrade filter fabric between flanges (C)
(d) caulking
(e) microporous filter mounting bracket
(f) microporous filter (secured in bracket A)

Fig.1. Permeability test apparatus.

than the largest beads. The shaking time for the sieve set is exactly 20 minutes. It is very important to use closely graded glass beads. At present, more than 90 per cent of the beads used for round-robin testing are in the range specified.

The US Army Corps of Engineers has established physical strength requirements for fabric used in riprap installations. The Corps says that its acceptability levels for fabric strength, listed below, may be reduced by 50 per cent if the fabric is to be used in drainage trenches or beneath concrete slabs, or if it is to be cushioned from rock placement with layers of sand or with zero drop height of rock placement. The physical strength of the fabric considered for use with drainage conduits will probably be tested through adaptations of two ASTM tests, D-1682 (Philadelphia, 1964) and D-751 (Philadelphia, 1973):

1. For tensile strength, ASTM D-1682 will probably be adapted for testing new fabrics. This is generally a grab test using 1-in^2 (645-mm^2) jaws and a travel rate of approximately 12 inches (305 mm) per minute. For this test the US Army Corps of Engineers specifies a minimum acceptance level of 200 lb (91 kg) in any principal direction.

2. Bursting strength will probably be tested with ASTM D-751. A diaphragm bursting tester will be used on new fabrics. The Corps' minimum acceptability level is 500 psi (3,488 kilopascals).

3. Puncture strength, like bursting strength, will probably be tested with ASTM D-751 on new fabrics. In the test procedure, a tension-testing machine with a ring clamp is used. The solid steel ball is replaced with a 5/16-inch (8-mm) diameter solid steel cylinder centered within the ring clamp. The Corps' minimum acceptability level is 120 lb (54 kg).

4. Abrasion resistance, like tensile strength, will be tested with ASTM D-1682 but after abrading as in the ASTM D-1175 rotary-platform, double-head method (1). Rubber base abrasion wheels are used. The Corps' minimum acceptability level is 55 lb (25 kg) in any principal direction.

Other appropriate test procedures - for chemical resistance, ultraviolet resistance, and tear strength - are being searched for. Round-robin testing of these procedures will also be needed.

Many subsurface drains in the United States are protected with various brands of filter fabrics. Unfortunately, less than 50 of these installations have been excavated for a through evaluation of fabric performance. These excavations have provided some information, however, and we have confidence in filter fabrics if the silt fraction (0.05 mm to 0.002 mm) in the soil around the conduit is less than 40 per cent. Most soils with a high clay content, particularly those with more than 50 per cent clay (0.002 mm and finer), do not require a filter to protect the drain because the soil is cohesive. In these soils, sand and gravel envelopes are often placed around the drainage conduit to improve inflow characteristics.

The goal of SCS and the task force is to develop a reliable procedure that can test a specific filter fabric to be used in a particular soil. This would allow site-specific testing for critical installations where such tests are necessary.

Reliable test procedures will also allow us to test ranges of soils and classify them for their general filter requirements in areas where many subsurface drainage systems are installed. In this way, we hope to reduce the chances of error that can result in closely failures of these systems.

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Paper 2.07

DEFINITION OF THE DRAINAGE FILTER PROBLEM AND A POSSIBLE USE OF SOIL CONDITIONERS

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Summary

The main requirements placed on drainage envelope materials are that they prevent soil particles from entering the drain and that the envelope itself does not become clogged. Thick and coarse material seems so far to be the best. For a cheaper solution, envelopes can be used.

A preliminary test indicates that a soil conditioner soaked into the envelope material may be useful. The theory of piping or soil detachment by flowing water is developed. The soil parameters for this theory were measured by an erosimeter. They are a critical hydraulic gradient that would detach an aggregate of a given size or pass it through a certain sized filter pore. Here too, the addition of a soil conditioner to the soil will reduce erosion and help maintain higher hydraulic conductivity.

Tasks of envelope materials around an underground drainage pipe

Envelope materials are applied for the following reasons:

1. To retain soil particles. Such particles may enter the drainage pipe and cause it to clog. For some sensitive structures it is important to prevent the sedimentation of soil transported by the drainage water. Piping is the result of unretained soil particles and can lead to the destruction of hydraulic structures.

2. To increase the effective permeable diameter of the drainage pipe. This may be achieved in two ways:

a) The first is by actually increasing the permeable diameter, by combining the pipe with a thick permeable envelope material. The radial flow resistance is thus reduced. Having the original ratios of pipe radius R_o and the enlarged one R_f, the ratios of radial head losses for a given discharge or the relation of hydraulic resistances are:

$$\frac{\text{radial resistance with envelope}}{\text{radial resistance without envelope}} = \frac{\ln(R_f/R)}{\ln(R_o/R)} \simeq 1 + \frac{\Delta/R_o}{\ln(R_o/R)} \qquad (1)$$

where R is some dimension of the radial flow or an equivalent length of the radial part of the hydraulic resistance, and Δ is $R_f - R_o$, the thickness of the envelope material.

Equation (1) could be obtained because the hydraulic resistance for an ideally symmetric steady radial flow is:

$$\frac{\Delta\phi\text{rad}}{Q} = \frac{\ln \frac{R_o}{R}}{2\pi K} \qquad (2)$$

where K is the hydraulic conductivity, Q the discharge per unit length of pipe or well, and $\Delta\phi$ is the radial head loss. The radial approximation is obtained by extending the numerator in Eq.1 to

$$\ln \{(\Delta/R) + R_o/R\}$$

and by Taylor's expansion series.

A typical value of the denominator in Eq.2 is minus 4 or 5. Thus a 50% increase in R_o gives only a 10% reduction in the radial resistance. As the radial resistance itself is only part of the total flow resistance towards a drainage pipe, this improvement is not very effective.

b) The second form of increasing a drainage pipe's effective permeable diameter and of reducing the resistance to flow is by increasing the pipe's effective perforation almost to infinity. The local contraction of streamlines towards a pipe perforation is over a distance that is of the same order of magnitude as the distance between perforations (Kirkham and Schwab, 1950 and 1951). A thick permeable envelope can produce almost an ideal perforation or zero entrance resistance. (Nieuwenhuis and Wesseling, 1958.) The effect around corrugated plastic pipes is different from that around smooth pipes. The envelope bridges the corrugations and can form a continuous finely perforated surface even when it is very thin. Without the corrugations the envelope must be several millimeters thick to allow for easy lateral flow towards holes in the pipe's circumference.

3. To integrate or hydraulically short-circuit cracks, root holes, and other permeable elements in the ground. The larger the perimeter of the filter, the greater is the probability of such integration.

4. To provide indirect connections between laterals and main pipes.

5. To short-circuit partially clogged drains, letting the water bypass the clogged part.

6. To increase the hydraulic conveyance capacity of the pipe.

The main uses are the first two. With the present prices of pipes and filters it will not pay to increase permeable diameter by a thick filter. A cheaper solution is to increase the diameter of the pipe itself.

The cost per metre of larger diameter drainage increases, whether the diameter enlargement is obtained by a larger pipe or a thicker filter. The alternative is to install a denser drainage system, which has added benefits as well. There must be an optimum between the alternative but it is difficult to calculate. The minimum is a very small pipe diameter. The actual minimum diameter seems to be determined by maintenance requirements. Higher diameters are determined by hydraulic conveyance requirements.

Thus we are left with two possible uses of the filter:

- holding back soil particles, and
- approaching an ideal continuous perforation.

Materials tried for filters

Numerous materials have been tried as envelope materials. Several papers in the Workshop report on them. For our purpose, a simple classification is:

1. *Gravel*. Gravel has been used more than any other envelope material, and usually with success. It is rather expensive, almost doubling the price of drainage.

Gravel envelopes have all the advantages of bulky filters. In some heavy soils they are absolutely essential. The reason for this is that the drain must be laid below the plough layer while the soil at this depth tends to be absolutely impermeable. The gravel then connects the drainage pipe with the plough layer.

2. *Artificial aggregates produced from soil* (Dierickx and Goossens, 1979). Artificial aggregates have been produced by using Portland cement, lime, asphalt emulsions, and polymer resins of various kinds. Probably the cheapest among them would be those with Portland cement or lime. At about 5 per cent level by weight of Portland cement, there must be a ratio of 1:20 between the cost per unit weight of a natural aggregate and per unit weight of cement to have the same cost. In most cases artificial aggregates will not be economical as a substitute for natural gravel, volume per volume, although they may have some merits if some special technique could apply them in a smaller quantity around the drainage pipe. They may also be of interest when gravel is scarce and expensive or when it is too coarse to prevent soil particle transport.

3. *Fibrous envelopes*. Fibrous envelopes are made of natural materials such as coconut fibre or peat, or of synthetic materials. There are many examples of glass, plastics and other fibres. The main experience with these envelopes can be summarised as follows.

a) *Fine structured* envelopes tend to clog by clay accumulation on their surface and possibly by deposition of organic matter and chemicals.

b) *Thin coarse structured* envelopes do not fulfil their task because they let particles of silt enter the drain. Such particles will easily settle and will not be washed out.

c) *Thick and coarse structured* envelopes have worked well. There is no proven theory for the above findings, but an attempt will be made here to explain them.

Why thick and coarse structured envelopes work?

The theory of detachment of soil particles has been formulated and demonstrated elsewhere (Zaslavsky and Kassiff, 1965).

Particles smaller than the envelope's pores leave the soil. Larger particles remain behind. This process may continue until the soil stabilizes and an inverted natural soil filter gradually forms, changing from coarse to fine grains. (See appendix). If the envelope is thin and too coarse and the hydraulic conditions are appropriate, the erosive process may continue almost without a chance of stabilization. If the envelope is thin and fine, all the small particles will be stopped at one plane or within a thin surface and will soon clog it. With a thick and coarse envelope, however, the fine particles leaving the soil will be stopped somewhere within the envelope at different depths. The probability of the envelope filling up completely over a continuous surface and thus becoming clogged is reduced. The inverted natural soil filter has an opportunity to develop before the clogging gets too serious. It then stabilizes the soil against further erosion while somewhat increasing the effective thickness of the envelope.

Alternative approaches to produce an envelope

A thin and coarse envelope would be the cheapest. Such an envelope, however, would be effective only if the soil behind it had large enough particles or aggregates to form the inverted soil filter with little silting into the drainage pipe.

Alternatively, a finer structured envelope could be used if there were no free fines in the soil to migrate and clog the filter. The bonding of clay and silt particles into stable aggregates enables the use of a thin envelope.

This is really the experience in many clay soils with high cohesion and stable aggregates. An envelope is then rarely required.

Where natural cohesion is lacking, the use of a soil conditioner can be tried. The use of conditioners differs from the approach of producing a bulk of soil aggregates around the pipe. It is necessary to treat only a relatively thin layer of soil around the drain. The requirement of the envelope material can now be formulated as the elimination of soil particles smaller than some 50 or 100 microns. In other words, the requirement is not to produce a bulk of large aggregates around the drain but to prevent the clogging of fine structured envelopes or the failure of coarse and thin envelopes.

Preliminary experiments

Test results have been obtained with a soil conditioner called "Lima". It is a soluble sodium salt of the copolimer of lignosulfonates and acrylic monomers. Its application to soil at a level of one part per thousand produces over 70 per cent water stable aggregates larger than 0.1 mm in a loess soil that has naturally only 36 per cent of such aggregates. (See appendix 1).

Another possibility of applying the solution of "Lima" is in association with trenchless drainage ploughs. An injection or spray of the stabilizer solution could be applied around the cavity produced to contain the drainage pipe.

Experience with a vigorous spray on a soil surface in a high water dilution showed a stabilization effect to a depth of a few millimetres.

Appendix 1

Test results with a soil stabilizer

1. Typical results of wet sieving after treatment with various stabilizing agents (see Tab.1).

2. Typical results for clay soil Nes Amim-Israel by wet sieving

	0.5% Na-Lima 1100	no treatment
larger than 2 mm	37.6 %	6.5%
1-2 mm	32.7 %	15.5%
0.5-1 mm	18.5 %	35.8%
0.25-0.5 mm	6.38%	21.9%
larger than 0.25 mm	95.44%	81.6%
0.1-0.25 mm	4.93%	14.7%
larger than 0.1 mm	100.4 %	90.2%
average weighed diameter (mm)	1.24	0.575

3. Experiments with 0.1 per cent Na-Lima 1100 were conducted in stabilized Loess with two types aggregates 0.84-2 mm and 0.42-0.84 mm. In neither case was there any obvious settlement of the aggregates in the permeameter after wetting. There was 10 per cent settlement in the unstabilized soil.

The respective permeabilities to air were related to the permeability to water (k_a/k_w).

Aggregate size	Permeability Ratios		
0.84 - 2 mm untreated	(k_a/k_w)	=	675 - 386
0.84 - 2 mm stabilized	(k_a/k_w)	=	110 - 29
0.42 - 0.84 mm untreated	(k_a/k_w)	=	64 - 40
0.42 - 0.84 mm unstabilized	(k_a/k_w)	=	27 - 16

Clearly the stabilized soil maintains hydraulic conductivity that can be 2.5 to 12 times larger than the untreated soil.

TABLE 1. The soil: Loess of Northern Negev (clay 38%, silt 49%, sand 13%)

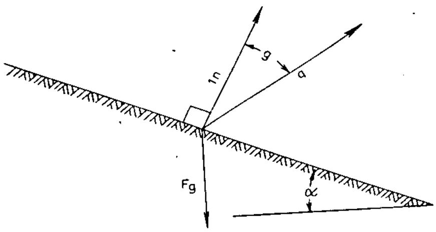
% of stabilizer by weight	0.025		0.050		0.075		0.100		0.200	
	% stable aggregates by weight									
	larger than 0.25 mm	larger than 0.1 mm	larger than 0.25 mm	larger than 0.1 mm	larger than 0.25 mm	larger than 0.1 mm	larger than 0.25 mm	larger than 0.1 mm	larger than 0.25 mm	larger than 0.1 mm
Type of stabilizer										
Water alone	14.8	36.2								
Krilium	19.5	39.2	21.6	42.3	23.4	44.7	28.1	49.3	32.1	53.4
Cat flock			38.5	49.5	45.0	56.9	47.5	61.2	51.0	74.9
Na-Lima 1100	31.1	54.3	44.6	68.5	47.4	67.0	53.5	73.1		
Portland cement	0.50% by weight						1.0% by weight		2.0% by weight	
	27.5	48.6					32.5	55.0	42.5	66.6

Appendix 2

Stability of soil fragments against seepage forces

The movement of soil particles by flowing water is governed by seepage drag forces. The theory that explains this mechanism is the following:

1. *Piping of non-cohesive material.* Consider a soil surface making an angle α with the horizon or slope $m = \tan\alpha$ (Fig.1). Consider also a unit vector normal to the soil surface and pointing out of the soil. A flux



q = water flux vector

F_g = submerged weight of soil fraction

ρ = vanishes in isotropic soil with equipotential soil surface

Fig.1. Outflow seepage from sloping surface.

vector q makes an angle with this unit vector. Assuming moderate head differences in the water above the soil surface, equipotentials will be parallel to the soil surface. (This will not be the case with a thin water layer flowing downhill, the so-called seepage face). The hydraulic gradients will be orthogonal to the soil surface. Thus, any angle ρ between q and $1n$, the normal unit vector, will only be found in anisotropic soil. For the sake of simplicity, only the isotropic and orthogonal case will be treated. The general case is then straightforward. The net submerged weight of a particle (for the total volume including pores) F_g is as follows:

$$F_g = -(1 - n)(\gamma_s - \gamma_w)\ V1z \tag{2.1}$$

where γ_s is the unit weight of the pore-free solid material, γ_w the unit weight of soil solution, V volume of the soil fragment, $1z$ unit vector in an upward direction, n is the porosity, z elevation.

The component normal to the soil surface (direction of 1n) is

$$F_{gn} = |F_g| \cos \alpha = - (1 - n) (\gamma_s - \gamma_w) V \cos \alpha \tag{2.2}$$

The seepage force (assuming orthogonality) is

$$F_{sn} = - V \gamma_w \text{ grad } \phi; \; \phi = \rho/\gamma + z \tag{2.3}$$

Thus, a flow out of the soil has a positive flux q and a negative gradient grad ϕ, and F_{gn} is positive. Combining F_{gn} of Equation 2.2 and F_{sn} of Equation 2.3, the total active force is

$$F_a = - V\{\gamma_w \text{grad } \phi + (1 - n)(\gamma_s - \gamma_w) \cos \alpha\} \tag{2.4}$$

In a cohesionless soil this force must be positive to cause piping. In other words, the condition for piping is that:

$$\frac{\gamma_w \text{ (grad } \phi) \text{ (1n)}}{(1 - n) (\gamma_s - \gamma_w)} \cos \alpha > 1 \tag{2.5}$$

This is a generalization of the commonly presented piping formula for horizontal soil surface (usually called boiling or quicksand). Several obvious conclusions may be drawn here:

a) An outward flow (grad $\phi < 0$) may cause piping. Infiltration (grad $\phi > 0$) is a stabilizing mechanism.

b) On a steep slope ($\cos \alpha < 1$) the conditions are less stable against piping. Here the stability of a single aggregate is considered, regardless of the possibility that the slope as a whole may become unstable at α approaching the internal friction angle. It is realistic to consider larger values of a α only if there is an incoming seepage that acts as a stabilizer or if some other processes such as electro-osmosis are being used for stabilization.

c) Under the same gradients and cohesion, a compacted material (small n) will be more stable).

d) Any mechanism increasing the outward gradient or decreasing the inward gradient will decrease stability against other forces such as drag by flowing water, splashing by raindrops, earthquakes etc.

One of the more significant conclusions can be drawn by substituting grad ϕ in Equation 2.5 by (q/k). If the conductivity is low, the piping can occur under an extremely small flux of water. Furthermore grad ϕ may be larger over an extremely small soil volume (cavitation point) and piping will occur under an extremely small water discharge. This is in line with some observations.

2. *Stability against piping in a cohesive soil.* In Equation 2.4 the gravity, flotation, and seepage forces were summed up. If there is net force F_a that tends to detach the particle from its place, an adhesive force F_c will develop as a reaction. Let us assume the maximum average tensile stress T between aggregates and a contact area A (without any momentum)

$$F_c = a\ T\ A \tag{2.6}$$

where a is some geometric coefficient and A a surface area of this soil fragment. The direction of F_c is always colinear and opposite to the net force in Equation 2.4.

For a soil fragment to be unstable, the criterion is now

$$- V\{\gamma_w \text{ grad } \phi + (1 - n)(\gamma_s - \gamma_w) \cos \alpha\} > a\ T\ A \tag{2.7}$$

Rearranging Equation 2.7 and putting (V/aA) = bD where D is an equivalent particle diameter and b a geometric coefficient, one gets as a criterion for instability

$$- \frac{bD}{T} \{\gamma_w \text{ grad } \phi + (1 - n)(\gamma_s - \gamma_w) \cos \alpha\} > 1 \tag{2.8}$$

To somewhat simplify Equation 2.8 we note an outward gradient by $-j$ = grad ϕ. An aggregate will be unstable if

$$\frac{bD}{T} \{\gamma_w j - (1 - n) (\gamma_s - \gamma_w) \cos \alpha\} > 1 \tag{2.9}$$

If the outward flux q is known (assuming an isotropic equipotential soil surface), then in place of Equation 2.9 we can write

$$\frac{bD}{T} \{\gamma_w \frac{q}{K} - (1 - n) (\gamma_s - \gamma_w) \cos \alpha\} > 1 \tag{2.10}$$

Many heavy soils may develop tensile strength of up to $T = 0.1$ kg/cm^2 and when compacted even as high as 1 kg/cm^2. For the following estimate, one can consider b as being around unity. Clearly in Eq.2.9 with $T = 0.1$ kg/cm^2

$$\frac{bD\gamma_w j}{T} >> \frac{bD}{T} \{(1 - n)(\gamma_s - \gamma_w) \cos \alpha\} \tag{2.11}$$

And therefore for highly cohesive soils the criterion for stability of a fragment against piping is approximated by

$$\frac{b}{T} D\gamma_w j < 1 \tag{2.12}$$

The neglected term in Equation 2.9 is $(1 - n) (\gamma_s - \gamma_w) \cos \alpha$.

If $T = 0.1$ kg/cm^2, then the product Dj must be of the order of 10^2. Clearly the neglected term, which is at best of the order of unity, is negligible. It is interesting to note that, for the particle diameter $D = 0.1$ cm, the hydraulic gradient j must be of the order of 1000 to cause piping. This is what was actually found in experiments (Zaslavsky and Kassiff, 1965). It explains why, in cohesive soil, splashing by raindrops or free swelling and dispersion are necessary to produce appreciable erosion. The momentary outward gradients developed by a raindrop can be very high. A highly dispersed swollen clay has a lower T.

Evidently, large soil portions will be more easily washed out because of larger D and smaller T. This is really the experience in channels through cohesive soils, where often large chunks of soil fall out from the bank into the water stream. For large soil portions, however, high hydraulic gradients can seldom be maintained.

The actions of a soil conditioner lies either in increasing the diameter or in increasing the cohesion for a given aggregate diameter.

3. *Test of piping.* Erosimeters were installed in the form of cylinders containing soil samples. The soil sample was supported by a screen of a given hole size or by a plate with holes of given sizes. Water was then allowed to flow through the soil, which tended to detach soil particles through the bottom screen. The hydraulic gradient was gradually increased until soil particles started falling out of the screen into a transparent water jar.

In trying glass beads, it was found that the beads will fall continuously through if the hole sizes are some 50 per cent. With smaller holes, the beads bridge to stop piping. With natural soil particles, the bridging effect can occur over much larger holes.

A few per cent of larger beads will eventually stop outfall of smaller beads unless the beads are shaken and remixed.

When cohesive soil was placed in the erosimeter, the first erosion occurred at local cavitation points at the edges of holes or screen wires. The soil then tended to form small dome-shaped cavities that had an evened-out, smaller outward flow gradient. A further increase of the water head eventually started a progressive piping that would not stop of its own accord. The critical gradient J and the hole size are characteristics of the soil and can be used for design purposes.

An additional effect is a local compaction of the soil under the hydraulic head differences. The result is a reduced hydraulic conductivity.

The use of a soil conditioner increases the resistance to large water head and reduces both piping and compaction.

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Paper 2.08

THE ENTRANCE RESISTANCE OF DRAINS AS A FACTOR IN DESIGN

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Summary

In design of drainage systems, the entrance resistance of the pipe system is generally neglected, although in material research quite some emphasis is laid on this aspect. Small diameters of pipes and high entrance resistance lead to submerging of drainage systems. With the aid of solutions for the flow problem in drainage it is explained what effect submerging may have on the drainage system applying Ernst's theory.It is proved that by submerging the radial resistance of the system decreases. Arguing that a decrease in radial resistance can be used to allow a certain entrance resistance, some simple formulas have been developed to take into account the entrance resistance. The results shown for entrance resistance of pipes with filters are generally such that they have no effect on the drain spacing. This does not hold for very thin filters, especially when smooth drainage pipes are used.

Introduction

Drainage formulas (Hooghoudt, 1940; Kirkham, 1958) normally used for design purposes, have been derived for so-called "ideal drains". No doubt the present drainage materials do not form an "ideal drain" but merely have a certain resistance against flow of water through their walls. The question to be answered is, whether this entrance resistance may cause a reduction in drain spacing so that the designed system fulfils the aims set to it. People working in the practice of drainage design often propose to reduce the available head taken into account in the design, with a certain percentage to overcome the entrance resistance. This method would then lead to a narrower drain spacing. A further reasoning is that material with a high entrance resistance, causing narrower spacings, then can be compared with material with lower resistances by computing the additional costs due to the reduction in drain spacing.

The above reasoning, however, is not correct. Introducing a certain entrance resistance means that there should stand water above the drains in other words the system should be submerged. Drainage theories (Kirkham, 1958; Childs and Youngs, 1958; Dagan, 1964; Engelund, 1951; van Deemter, 1950; Wesseling, 1964) clearly show that submerged drains are liable to more favourable flow conditions than the cases for which the drainage formulas have been derived. This implies that part of the reduction in spacing introduced by taking into account an extra loss of hydraulic head should be cancelled against the more favourable flow conditions.

In this article a solution for the problem discussed above will be discussed. For this purpose the flow theories will be treated briefly to explain what factors should be taken into account. Next the effect of the entrance resistance on spacing will be discussed. Finally computations will be carried out to show what magnitude the entrance resistance may have before it will have any effect on the required drain spacing.

The ideal drain

Many solutions for drainage problems have been given in literature. For our purpose let us consider van Deemter's solution for flow towards parallel drains with an impermeable layer at infinite depth (Fig.1) obtained by the hodograph method.

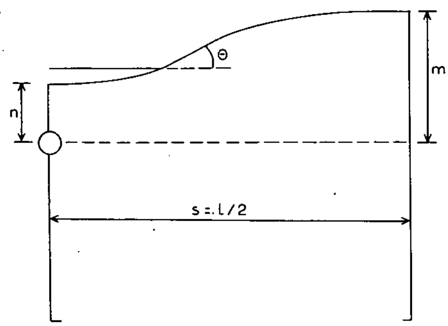

Fig.1. Schematic diagram of the flow problems solved by van Deemter (1950).

Van Deemter arrives at the general solution

$$\frac{\pi m}{s} = \ln \frac{2 + \beta}{\beta} + \frac{2}{\gamma} \ln \frac{2 + \beta}{2} \tag{1a}$$

$$\frac{\pi n}{s} = \ln \frac{2 + \beta}{\beta} + \frac{2}{\gamma} \ln \frac{2+\beta}{2 + 2\beta} \tag{1b}$$

with $$\gamma = \frac{k + R}{S - R} > 0 \tag{1c}$$

where

k = hydraulic conductivity (m/d)
s = half the drain spacing (m)
R = rainfall intensity = discharge (negative!)(m/d)
S = underground seepage intensity (m/d)
m = hydraulic head midway between drains (m)
n = hydraulic head above drains (m)

The factor β represents the pressure in the drains. It comes from the transformations.

$$1 + \beta = \mu/\lambda \tag{2a}$$

$$\frac{\mu^2 - 1}{2\mu} = (1 + \gamma) \tan \theta \tag{2b}$$

$$\frac{\lambda^2 - 1}{2\lambda} = \tan \theta \tag{2c}$$

where θ is the largest angle of the water table with the horizontal plane. In the case of optimal drainage, i.e. the lowest possible water table

$\theta = \frac{\pi}{2}$ and $\beta = \gamma$ so that

$$\left(\frac{\pi\ m}{s}\right)_{min} = \ln \frac{2 + \gamma}{\gamma} + \frac{2}{\gamma} \ln \frac{2 + \gamma}{2} \tag{3a}$$

$$\left(\frac{\pi\ n}{s}\right)_{min} = \ln \frac{2 + \gamma}{\gamma} + \frac{2}{\gamma} \ln \frac{2+\gamma}{2 + 2\gamma} \tag{3b}$$

which is similar to the solution of Engelund (1952).

When the drain is just running full, $n = r_o$. So for various values of γ the value $\pi r_o/s$ can be computed. If one takes S = 0 then

$$\beta = \gamma = \frac{k + R}{- R} = - \frac{k}{R} - 1 \tag{4}$$

For fixed values of k/R the corresponding value for r_o/s = d/L can be found. This relation is indicated in Fig.2 as the curve for H/L = ∞. In this figure H is used for the depth of the impermeable layer, L for drain spacing and d for the diameter of the drain.

List (1974) developed an approximate solution for the drainage problem with an impermeable layer at finite depth. The curves showing the relation between d/L and R/k for H/L = 0.15, 0.20 and 0.25 in Fig.2 are those computed by List.

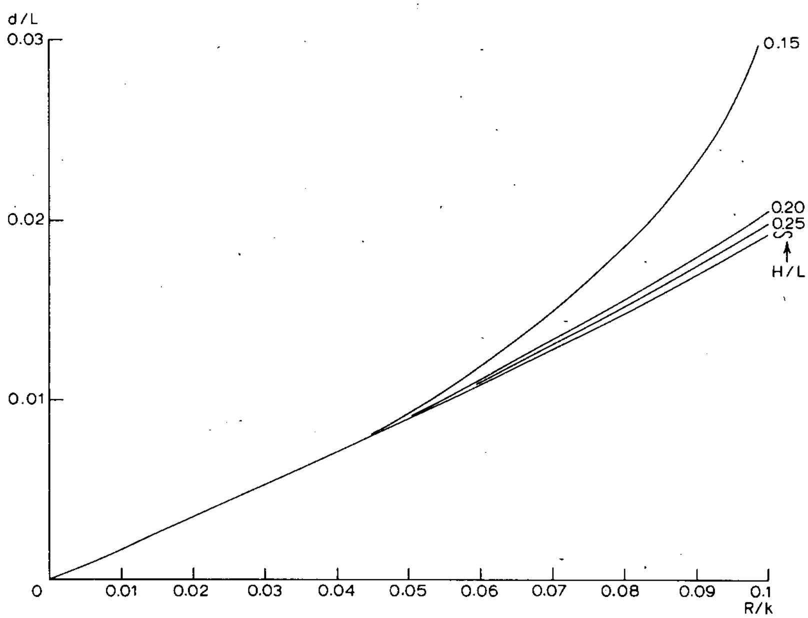

Fig.2. Relation between d/L and R/k for various values of H/L (after LIST, 1974).

Fig.2 now gives the conditions for optimum drainage. Suppose for instance k = 0.4 m/d and H = 2 m, a drain diameter of 65 mm and a drain depth of 1 m. According to the drainage criteria used in The Netherlands these data give a drain spacing of L = 18 m. Fig.2 gives for d/L = 0.065/18 = 0.0036 optimum drainage conditions for R/k = 0.02. So if the discharge is larger than R = 10 mm/d there will be water standing above the drain.

On the other hand when $R/k = 0.007/0.5 = 0.014$ the figure shows that d/L must be 0.0026 or $d = 0.0026 \times 18 = 0.047$ metres in order to have no water above the centre of the drains.

Along the latter line it is possible to take into account entrance resistances of drains by introducing the concept of the equivalent drain radiu[s]. This concept has been worked out by Youngs (1974) and Nieuwenhuis and Wessel[-]ing (1978). The effective diameter of drain pipes and filter materials used is so small, that in most practical cases the conditions of optimum drainage will not be fulfilled.

The question then remains what influence this fact will have on the design of a drainage system. Let us therefore consider the consequences of water standing above the drains.

Submerged drains

Referring to Fig.3 a drain is called here submerged when n is larger than r_o. The effect of submerging has been described by several authors. Both van Deemter (1950) and Childs and Youngs (1958) show that when n increases, the hydraulic head m midway between the drains does not grow proportionally.

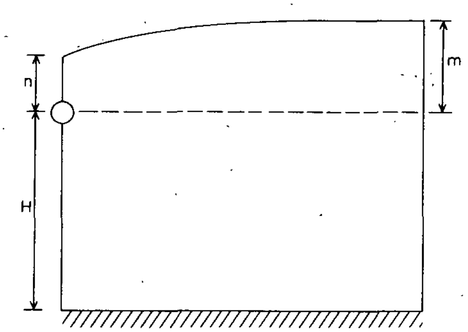

Fig.3. Schematic representation of a submerged drainage system.

In both publications the hodograph theory is used so that the maximum angle θ of the water table (see Fig.1) must be known or assumed. The analytical solutions of Kirkham (1958) for submerged drains offers the possibility of comparing the relation between m and n directly without taking into account the actual shape of the water table.

For our purpose we may put Kirkham's solution in the form

$$m - n = \frac{R\ L}{k} F \left(\frac{n}{H + n}, \frac{H + n}{L}\right) \tag{5}$$

The values of F have been computed by Wesseling (1964). Since the tables in that article have some small errors due to non-permissible simplifications in the equations used, a revised table is given here.

TABLE 1. Values of F according to Eq.5 derived from Kirkham's theory

$\frac{(H + n)}{L}$ \ $n/(H + n)$	0.2	0.4	0.6	0.8	1.0
0.01	12.65	12.45	12.35	12.30	12.28
0.02	6.403	6.199	6.097	6.045	6.029
0.04	3.278	3.074	2.972	2.920	2.904
0.08	1.716	1.511	1.409	1.358	1.342
0.16	0.934	0.730	0.628	0.577	0.561
0.32	0.540	0.338	0.239	0.190	0.175
0.64	0.307	0.130	0.059	0.031	0.023
1.00	0.186	0.052	0.015	0.005	0.002

From the table it can be seen that with increasing values of n/(H + n) the values of F decrease, so the difference between m and n. This effect is the larger the more (H + n)/L increases.

Dagan (1964) developed an approximate solution for the same problem which comes in the following equations

$$(H + m)^2 - (H + n)^2 = \frac{R\ L}{k} \left[\frac{L}{4} - \alpha\ (H + N)\right] \tag{6a}$$

$$\alpha = \frac{1}{\pi}\ \ln \left[2(1 + \cos \frac{\pi\ H}{H + n})\right] \tag{6b}$$

The results of this solution are nearly identical to those obtained from Eq.5. Therefore Eq.6 was used to complete values of m and n for the example used above. The results are given in Table 2.

TABLE 2. Computed values for m and n

a*		b*	
m	n	m	n
0.47	0.00	0.47	0.00
0.49	0.05	0.48	0.05
0.49	0.10	0.48	0.10
0.51	0.15	0.49	0.15
0.53	0.20	0.52	0.20
0.56	0.25	0.55	0.25
0.60	0.30	0.59	0.30

* For case a: L = 18, k = 0.5, H + 2.0, r_o = 0.0325
For case b: L = 20, H = 4.0
Discharge R in both cases is set equal to 7 mm/d

The table clearly shows that with n increasing to about 0.10 metres, m does not grow in the same order.

Ernst (1962) developed a drainage theory based on the concept of resistances. According to this theory the solution for a homogeneous soil profile is given in the form

$$m - n = RL\ (w_h + w_r) \tag{7}$$

where w_h stands for the horizontal and w_r for the radial resistance. The resistance values for the optimal drainage conditions are

$$w_h = \frac{L}{8\ kH} \tag{8a}$$

$$w_r = \frac{1}{\pi\ k}\ \ln\ \frac{H}{\pi r_o} \tag{8b}$$

In case of submerging these values change into

$$w_h = \frac{L}{8\ k\ (H + n)} \tag{9a}$$

$$w_r = \frac{1}{2\ \pi\ k} \left[\ln \frac{H + n}{2\ \pi\ r_o} - \ln \sin \frac{\pi\ n}{H + n} \right] \tag{9b}$$

Comparing Eqs. 8 and 9 it is evident that in case of submerging the horizontal resistance will decrease due to the fact that the denumerator of 8a changes from H to (H + n). The radial resistance, however, will decrease faster as is shown in Table 3.

TABLE 3. Values of n, m and w_r for the example* of Table 2

n	w_r	m
0.00	1.89	0.52
0.05	1.55	0.52
0.10	1.35	0.54
0.15	1.24	0.57
0.20	1.16	0.60
0.25	1.11	0.64
0.30	1.06	0.68

* H = 2.0, k = 0.5, r_o = 0.0325, L = 18 following from Eqs.7,8 and 9

Also here m does not increase appreciably as long as n is not larger than about 0.10 to 0.15 metres.

From all the considered theories it is clear that a certain height of the water table above the centre of the drains does not influence the height of that water table midway between drains. Since the latter is used in design, a certain submerging will have no influence on the computed drain spacing.

The entrance resistance

Since a certain degree of submerging does not influence the height of the water table midway between drains, the problem still should be solved, what magnitude the entrance resistance, causing the sumberging, should have in order to give no influence on the required drain spacing. For this purpose the theory of Ernst may be used. The entrance resistance can be taken into account simply by adding a term w_e to Eq.7, so that the general equation becomes

$$m - n = RL(w_h + w_r + w_e) \qquad (10)$$

The reduction of the radial resistance due to submerging now can be expressed as by the ratio w_s/w_n where s stands for sumberging and n for normal. From Eqs.8b and 9b now it follows that

$$\frac{w_s}{w_n} = \frac{\ln \frac{H + n}{2 \pi r_o} - \ln \sin \frac{\pi n}{H + n}}{2 \ln \frac{H}{\pi r_o}} \qquad (11)$$

Fig.4 gives this ratio as a function of n/H and different values of r_o/H.

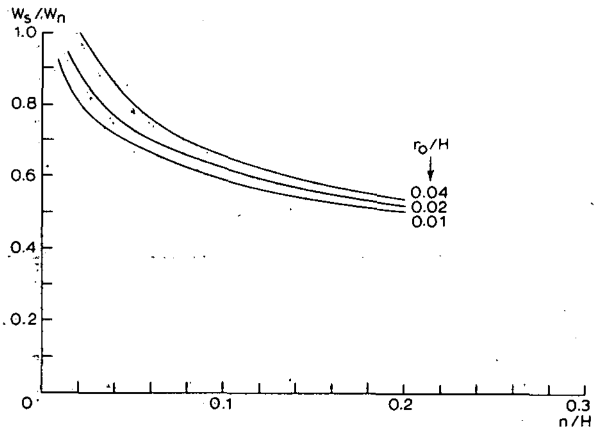

Fig.4. Graph of Eq. (4).

As could be expected, the radial resistance in case of submerging decreases as n/H is increasing. For larger drains (r_o/H larger) the effect is greater than for smaller drains.

Apparently a certain degree of submergence may occur before there is any appreciable effect on the height of the water table midway between drain In terms of Ernst's theory we may state that with submerging the radial resistance of the system decreases. Accepting now that a certain degree of sub merging does not influence the water table midway between drains the decreas in radial resistance may be used to allow a certain entrance resistance. For design purposes no difference in spacing then will be found as long as the ratio $w_s/w_n = 1$.

Let us therefore take a submerging of say 0.2 and a drain with r_o = 0.04 (60 mm pipe plus 1 cm filter). For H + 2 we have n/H = 0.1 and r_o/H = =0.02. From Fig.4 then w_s/w_n = 0.62. This means that with 0.2 metres water above the drain the radial resistance is reduced to 0.62 times its normal value and hence the entrance resistance may be up to 0.38 w_n before it has to be taken into account in the design.

In order to be able to compute the permissible entrance resistance we then may take

$$w_e = (w_s - w_n) \tag{12}$$

where w_s and w_n stand for the radial resistance in the submerged and normal case respectively. For these values Eqs.9b and 8b can be substituted, hence

$$w_e = \frac{1}{\pi k}\left[\tfrac{1}{2}\ln\frac{H+n}{2\pi r_o} - \tfrac{1}{2}\ln\sin\frac{\pi n}{H+n} - \ln\frac{H}{\pi r_o}\right] \tag{13}$$

or

$$w_e k = \frac{1}{\pi}\left[\tfrac{1}{2}\ln\frac{(H+n)\pi r_o}{2H^2} - \tfrac{1}{2}\ln\sin\frac{\pi n}{H+n}\right] \tag{14}$$

With the aid of Eq.14 the permissible value of $w_e k$ can be computed for different values of r_o, H and n. Fig.5 gives the results of such a computation in case r_o = 0.04 (60 mm pipe with 1 cm thick envelope).

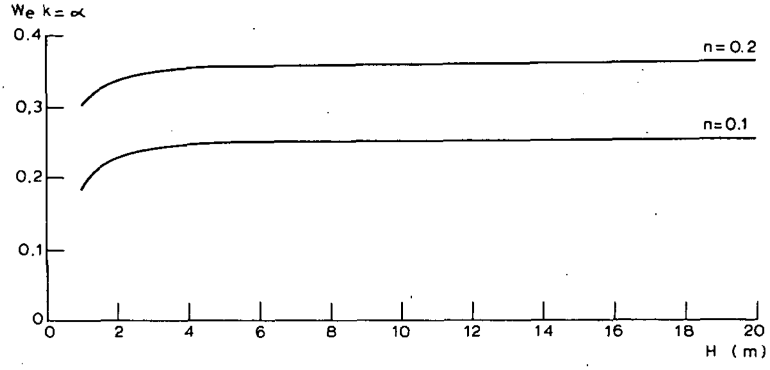

Fig.5. Permissible values of $w_e k$ for submerging depths n = 0.1 and n = 0.2 as a function of H for a pipe with r_o = 0.04 m.

The figure shows that over a wide range of H-values the value of $w_e k$ is about 0.36 for n = 0.2 and 0.25 for n = 0.1.

Actual entrance resistance values

In the preceding paragraph the permissible values for w_e have been discussed. The question now is what actual entrance resistances for drainage material can be expected.

Both from theory and laboratory experiments (Cavelaars, 1966; Wesseling and van Someren, 1972) it is found that the entrance resistance of drain pipes can be expressed in the general form.

$$w_e = \frac{\alpha}{k} \tag{15}$$

where k represents the hydraulic conductivity of the soils surrounding the drain. Now it follows from Eq.15 that $w_e k = \alpha$ so that the values given in Fig.5 represent the α-values defined in Eq.15. Wesseling and van Someren (1972) give a review of α-values derived from laboratory and field experiments on different types of pipes and filters of the IJsselmeer polders Authority. These data are given in Table 4.

TABLE 4. Values of α from field and laboratory experiments *

Type of drain	Filter	Field	Laboratory
Smooth PVC 5 cm	no	0.82	0.81
Smooth PVC 5 cm	glass fibre	0.40	0.14
PVC 5 cm	peat cover	0.08	0.011
PVC 5 cm	peat envelope	0.02	0.015
Clay pipe 5 cm	no	0.48	0.042
	no	0.30	0.46
	glass fibre	0.25	0.066
	peat cover	0.18	0.014
	peat envelope	0.01	0.005
Collar type clay	peat cover	0.05	0.017
Pipe 5 cm	peat envelope	0.02	0.004

* *Wesseling and van Someren, 1972*

Values found for corrugated PVC-pipes turn out to vary between 0.03 and 0.08 (see Wesseling and van Someren, 1972, Table 4).

Comparing the data given in Table 4 with those in Fig.5, it is obvious that smooth pipes without filter material have α values of such a magnitude that they may influence the design. As soon as filter material is applied, the α-value falls to a considerably lower level, except those for the thin glass fibre filters. For corrugated pipes the values are such that they are several orders of magnitude lower than those given in Fig.5 as a limit.

The conclusion therefore is that for the generally applied corrugated pipes with filter envelope no entrance resistance needs to be taken into account in the design. A problem not yet solved, however, is that due to blocking of the filter material by soil particles, the original α-value of a system can grow considerably. Nieuwenhuis and Wesseling found that due to a lower permeability of part of the filter the α-value can grow easily with a factor 5. Those results stress the importance of research on the problem of choosing the right filter for the right soil.

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Paper 2.09

EFFECT OF PERFORATION AND FILTER MATERIAL ON ENTRANCE RESISTANCE AND EFFECTIVE DIAMETER OF PLASTIC DRAIN PIPES

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Abstract

The flow towards drain pipes with and without filter was investigated by means of numerical solutions. Conformal mapping of the actual flow plane onto a plane with parallel flow allowed the development of a method to find the distribution of potentials around drain pipes. Solutions for two-dimensional flow were obtained. Although in actual cases three-dimensional flow will occur, the solution obtained may be used to find the influence of perforation pattern, filter thickness and filter conductivity on the effectiveness of the drain. Examples of the influence of the factors mentioned on the entrance resistance and effective diameter are given. Conditions in which the filter is partly blocked by soil material also were treated.

Paper 2.10

RESEARCH ON ENVELOPE MATERIALS FOR SUBSURFACE DRAINS

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Summary

In the last 20 years field drainage has evolved from a purely manual job into a highly mechanized and automated operation. Developments in machinery have stimulated the use of cheaper and labour-saving drain envelope materials.

These technological developments have been so rapid that the drainage engineers have not been able to keep their place as trail-blazers for industry and contractors. New techniques and materials introduced have been followed by laboratory and field investigations on their performance, instead of the reverse.

In the laboratory new materials are subjected to tests in sand-tanks. Two types of sand-tanks used in The Netherlands are described; the vertical, cylindrical tank of ICW[2] and the horizontal tank of RIJP[3]. Materials that have shown promising results are then subjected to further testing under field conditions. These testing procedures have some distinct disadvantages. One is the long time between the introduction of a new material and the "green light" for its practical application, and another is that the results are only valid for the test conditions.

For the development of improved drainage materials, it is essential that research be conducted to investigate and quantify those parameters which are important for an envelope's performance.

Two studies were conducted in 1976 at the ICW: one to analyse the effect of perforation and envelope material on the entrance resistance of drain pipes and the other to investigate the hydraulic gradients in the immediate vicinity of the drain under various degrees of drain "fill" and submergence.

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In 1976 a joint effort was started by ILRI and ICW to develop design criteria for the porosity, pore-size distribution, and permeability of envelope materials for different soil types.

The results obtained so far are insufficient in number and repetitions to enable any significant conclusions to be drawn. The trends and relations observed between particle size of the soil, poresize of the envelope, and the selective filtering and hydrological performance of envelope materials had to be proved valid and reproducible on a much larger scale. These provisional results are considered encouraging enough for the study to be continued along the lines proposed. The findings of this type of research have to be combined with knowledge on the structural stability of soils, on soil mechanics, and on hydrodynamics.

1. General

1.1 Developments in drainage materials in The Netherlands

Until the mid-fifties, almost all drainage in The Netherlands was installed by manual labour. The materials used were clay tiles with an inner diameter of 0.05 m and a segment length of 0.30 m. Whenever an envelope or cover was needed, bulky, loose material was applied. In common use were fibrous peat litter, heather in combination with straw, or straws of wheat and flax. Less used were gravel, lavalite, and shells.

Smooth rigid unplasticized PVC-pipes entered the drainage market in the late fifties, almost simultaneously with the large-scale introduction of drainage machines. The pipes were delivered in segments of 5 to 6 m; their diameters were 0.04 and 0.05 m and they had longitudinal saw-slots, 20-27 mm long and 0.6-0.9 mm wide, forty to a running metre. At the same time, synthetic envelope materials were introduced; glass wool, rock wool, styropor and fibreglass fleece.

For varying reasons few of these materials met the requirements; an exception was fibre-glass fleece which is still successfully applied in the southwestern part of The Netherlands.

Technology development in drainage was further enhanced in the mid-sixties with the introduction of corrugated unplasticized PVC drains. These strong flexible drains of good quality material were delivered in lengths of 150-200 m, coiled in rolls. They enabled and stimulated developments in laying techniques: an increase in laying capacity, automation of pipe laying and depth control, and a reduction in required manpower. Through these developments, drainage costs could be kept down despite rising costs of labor and materials. To save on the manual labor required for handling and placing loose materials around the pipe, envelope (or rather cover) materials were developed in band form. These striptype envelope materials were carried on the machine in rolls and were brought underneath and/or on top of the pipes by simply unrolling the coils.

Stip-type envelopes were either purely organic fibres or a mixture of organic and synthetic fibres and were either stuck onto paper or held together by a network of binding thread.

Of the many experimental products, the only ones that performed satisfactorily were fibrous peat on a nylon netting and straws of flax and some cereals strengthened with binding thread. Towards the late-sixties, coconut fibres, which were believed to be durable organic fibres, were introduced as material for drain envelopes. By this time, however, it was realized that covering only the top 2/3 of the outer perimeter of the drain would not provide sufficient protection against drainline sedimentation, especially in fine sandy soils. This problem was alleviated by factory-made total surrounds for corrugated pipes. So around 1972, one could choose between pipes prewrapped with coconut fibres, fibrous peat, flax and cereal straw.

Given the uncertainty of the durability of organic materials, synthetic materials (either manufactured fibres or processed waste material) were now and then offered as drain envelopes.

After the disillusionment with synthetic envelopes in the mid-sixties, the hesitation in admitting these envelopes for practical application has long persisted. But largely because of problems encountered in the durability of coconut fibre envelopes and the threatening scarcity of good quality fibrous peat, it was recently decided by the Government Service for Land and Water use to admit two of the most promising synthetic materials for practical application. These are:

(i) polystyrene granules, diameter range 2-6 mm, held around the pipe with a perforated plastic foil and

(ii) polypropylene fibres brought and held around the pipe in the same way as coconut fibres.

Less voluminous synthetic materials such as acrylic fibre mat, various types of polypropylene fibre mats, and others are still undergoing field testing. The non-voluminous fleeces, both woven and non-woven, are only applied incidentally.

1.2 Functions of envelope materials

When considering an envelope material for subsurface drains, one must first understand its functional requirements and the factors influencing its performance. Two more or less conflicting functional requirements can be distinguished:

- *Selective filtering function:* The material should prevent the entry of those soil particles which would otherwise cause sedimentation and clogging of the drains, or block the perforations or tile joints, or clog or block the envelope itself.
- *Hydrologic function:* The materials should maintain or create a highly permeable zone around the drain, thus improving the rate of water entry into the drain.

The term "selective" has been used intentionally, because the envelope should not be so finely structured that all soil particles are prevented from passing through. The material needs to have a certain quantity of pores coarse enough to permit the passing of clay and silt particles, because otherwise the material would become blocked or clogged in the course of time, and so not meet the second requirement of facilitating water entry. Owing to the limited entrance possibilities in a drainline, the flowlines towards the drain must concentrate towards the perforations, with the consequence of relatively high flow velocities and the transport of soil particles. By introducing a highly permeable zone around the pipe, the number of pore connections on the boundary of soil/envelope will increase appreciably, thus decreasing the hydraulic gradient and diminishing the danger of particle movement.

Whereas fine-structured materials are preferred for the selective filtering function, voluminous coarse-structured materials are better for the hydrologic function.

What functional requirement will be decisive for the composition of the envelope material will depend on the soil type (texture and structure), the "wetness" condition at the time of installation, and whether precipitation of complex iron, manganese, or magnesium compounds need to be feared.

This paper will not describe the factors that determine the use and choice of envelope materials. These factors were the subject of an article published by Knops and Zuidema (1977), of which an English translation is published by Knops et al. (1979).

2. Research on envelope materials

2.1 Introduction

Since the introduction of unplasticized PVC-pipes and the man-made bonded fibre-glass fabric a continuous stream of "probable" and "improbable" products have been offered by the fibre- and/or envelope processing industry as envelope materials for surbsurface drains.

Most of the products were either processed agricultural or industrial waste products, with the exception of a few, generally more costly, manufactured ones.

The manufacturers of these products, however, had little or no knowledge of the properties required of drain envelopes and were mainly interested in finding new markets for their products, or in increasing their turnover. What this course of events definitely did was to place a heavy burden on the drainage engineer and the supporting research institutions, as they had to continuously judge the practicality of the new materials. Although the drainage engineer had built up a fair knowledge of the functional requirements of a drain envelope, he still lacked the knowhow to quantify the crucial properties as a function of soil type. These properties might be: required thickness, porosity and its distribution, permeability, compressibility and durability.

With the knowledge acquired from experience, from laboratory and field experiments, and from evaluating field research, the drainage engineer is able to present guideliness on the practicality of the currently available materials (Knops et al., 1979). Progress would really be made, however, if we could avail ourselves of design criteria for fibrous envelope materials comparable with the design criteria developed in the USA for sand/gravel envelopes. Realizing this need, a reconnoitring research was undertaken by the International Institute for Land Reclamation and Improvement (ILRI) in cooperation with the Institute for Land and Water Management Research (ICW).

It is the objective of this paper to present the considerations which have led to a re-evaluation of the procedure, established more than a decade ago, for testing envelope materials on their practicality.

2.2 Testing methods

What means does the drainage engineer or researcher have at his disposal to sift the corn from the chaff when confronted with the amount of probable and improbable materials offered to him for judgement?

He will make a first selection through visual inspection on the basis of his experience. But the materials that will remain for testing under field conditions will still be too many. The researcher will then use laboratory techniques for a further selection. He will test the materials in sand tank models, simulating as closely as possible the groundwater flow processes in the immediate vicinity of a field drain.

These tests should provide consistent and reproducible results, which are then compared with those obtained on products of known performance under field conditions. At this stage, however, the results are not reliable enough to permit direct application of the materials in the field. Promising materials are further tested in experimental fields, after which conclusions are drawn on their practicality under actual field conditions.

Laboratory testing

To simulate the groundwater flow conditions in the immediate vicinity of a drain, various types of sand-tank models have been developed. The two types used to test envelope materials in The Netherlands will be described.

Vertical, cylindrical model

In Figures 1 and 2 the tank used at the ICW is presented. This sand-tank model is a typical example of concessions made towards easy handling and the acquisition of quick results.

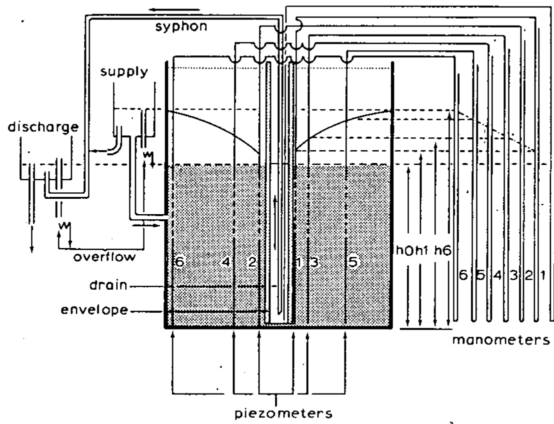

Fig. 1. Schematic representation of the vertical cylindrical sand tank.

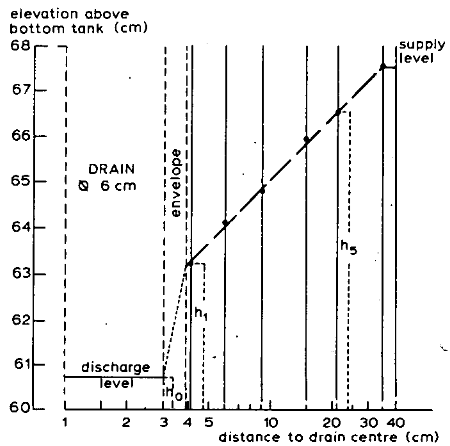

Fig. 2. Evaluation diagram for the hydraulic conductivity (k) and the entrance resistance (w_e).

The soil fill is a homogeneous moderately fine sand with 80-85 per cent of the soil particles in the range between 0.1-0.3 mm and a permeability in the range of 8-10 m/day.

The drain plus envelope is placed in the centre of a cylindrical container. The flow takes place from the outer boundary towards the drain and is discharged from the drain by a siphon. The flow in the model is horizontal and fully radial.

The Thiem-Dupuit formula for the flow towards a well that fully penetrates a phreatic aquifer is applicable to this model:

$$Q = \frac{\pi k \, (h_2^2 - h_1^2)}{\ln r_2 - \ln r_1} \qquad \text{for} \qquad \frac{h_2 - h_1}{r_2 - r_1} < 1/2 \qquad (1)$$

in which

- Q = discharge from the drain (cm^3/sec)
- k = hydraulic conductivity (cm/sec)
- h_1, h_2 = piezometric heads at distances r_1 and r_2 respectively from the centre of the well (cm) (see Fig.2)

From the discharge and corresponding piezometric head measurements, the permeability of the sand-fill can be calculated by means of Equation (1).

The entrance resistance of the drain pipe plus envelope can be derived from the water level inside the drain (h_o), the piezometric pressure just outside drain and envelope (h_1), and the flow per unit length of pipe (Q/h_o). (See also Figure 2.)

$$W_e = \frac{h_1 - h_o}{Q/h_o} \qquad (2)$$

in which

- h_1 = piezometric head just outside drain and envelope (cm)
- h_o = piezometric head inside drain (cm)
- W_e = entrance resistance in (sec/cm)

The flow resistance factor, α, being a dimensionless and characteristic value for the drain plus envelope, can be derived if the permeability (k) of the surrounding sand-fill and the entrance resistance (W_e) are known:

$$\alpha = W_e \, k \qquad (3)$$

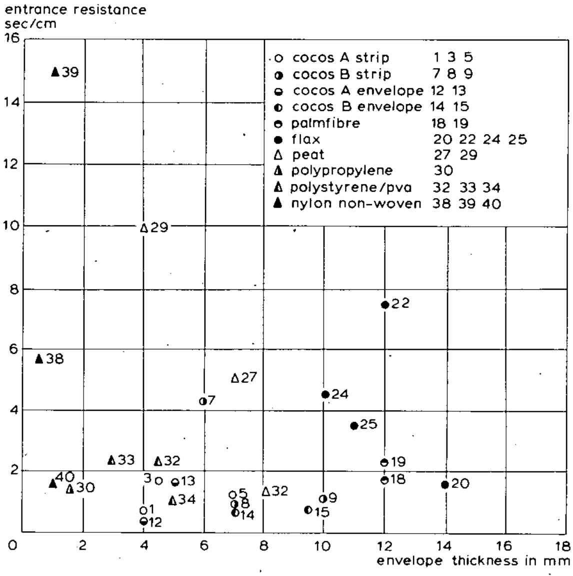

Fig.3. Entrance resistance plotted against thickness of the envelope material as tested in the vertical cylindrical sand tank.

In the past twelve years an impressive number of routine measurements have been conducted in this sand-tank model. Some of the results are presented in Table 1; while in Figure 3 they are plotted against the thickness of the envelope material.

> *NOTE: The W_e data presented in sec/cm can easily be converted into the dimensionless flow resistance factor, α, as the average value of the permeability of the sand-fill during the tests was 8 m/day. Thus multiplying by 8/864 or 1/100 makes the data comparable with the data reported by Wesseling and van Someren (1972).*

This type of sand-tank was chosen for routine measurements because of the following advantages:

TABLE 1: Entrance resistance, thickness, and sand-tightness of various tested drain envelope materials

Drain envelope material	Manufacturer	type and (gr/m^2)	Entrance resistance (sec/cm)	Thickness (mm)	Sandtightness*
A ORGANIC MATERIALS					
Coconut-fibres					
1 Strips	A	500	0.5	4	-
3	A	750	1.6	4.5	-
5	A	1000	1.0	7.0	-
7	B	500	4.3	6.0	-
8	B	750	0.9	7.0	-
9	B	1000	1.1	10.0	-
12 Envelopes	A	500	0.3	4.0	-
13	A	750	1.5	5.0	-
14	B	500	0.5	7.0	-
15	B	750	0.6	9.5	-
Palm-fibres					
18 Envelopes	C		1.7	12.0	xx
19	D		2.3	12.0	xx
Flax-straw					
20 Strips	A		5.2	14.0	x
22	E		7.5	12.0	-
24 Envelopes	A		4.5	10.0	x-
25	E		3.5	11.0	-
Fibrous peat					
27	E		5.0	7.0	-
29	F		9.8	4.0	-
B SYNTHETIC MATERIALS					
Polypropylene fibre mat					
30	G		1.4	1.5	-
Polystyrene flaxes in PVA/fibrous fleece					
32 5-10 mm	H	1 20	1.3	8	x-
33 2-5	H	2 30	2.2	3	-
34 5-10	H	3 40	1.0	5	-
Nylon non-woven					
38	I		5.6	0.5	x
39	J	140	15.0	1	-
40	J	70	1.4	1	-

* *Sand-tightness:* *very good* -- , *good* - , *moderate* x- , *weak* x , *very weak* xx

(Data provided by courtesy of Ing. H.J. Meyer of the Institute for Land and Water Management Research, Wageningen)

- The tank can be easily and rapidly prepared;
- It is easy to check on the manometer board whether steady state conditions have been obtained, as the piezometers are placed at logarithmic distances from the centre of the drain;
- The discharge rate can easily be changed and the model will react quickly because of its relative high permeability;
- Sediment entrance in the drain can be checked and measured;
- Full radial approach flow is obtained.

Some of the disadvantages of the tank are:

- Normal field soil, with permeabilities in the range of a few cm to to 1 m per day cannot be used as a soil-fill, apart from the difficulties one would have in obtaining consistent and reproducible results.
- The compression of fibrous organic or synthetic envelopes, which takes place under normal field conditions where soil pressures range from 0.2-0.3 kg/cm^2, is not simulated. This soil pressure would definitely exert its influence on the porosity distribution of fibrous materials.

Horizontal model

A distinctly different type of sand-tank model is employed by the Scientific Department of the IJsselmeer Polder Development Authority (RIJP).

The objective of the research done in this sand-tank is similar to that in the vertical, cylindrical sand-tank, i.e. to collect information on new materials in the shortest possible time and enable a judgement on their practicality.

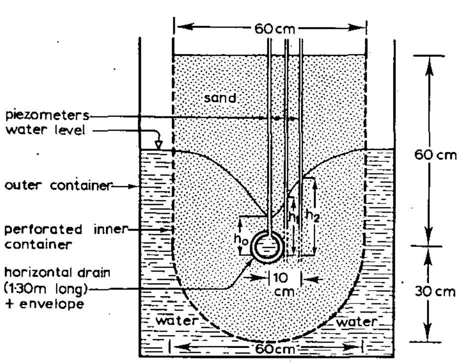

Fig. 4. Cross-section of the horizontal sand tank.

In this tank (a cross-section is presented in Figure 4) the drain is positioned horizontally, with the radial approach flow approximated by the U-shape of the inner container.

Similar to the vertical, cylindrical model, homogeneous sand is used as soil medium in order to obtain reproducible and consistent results. The particle size distributions of sand types used are presented in Figure 5.

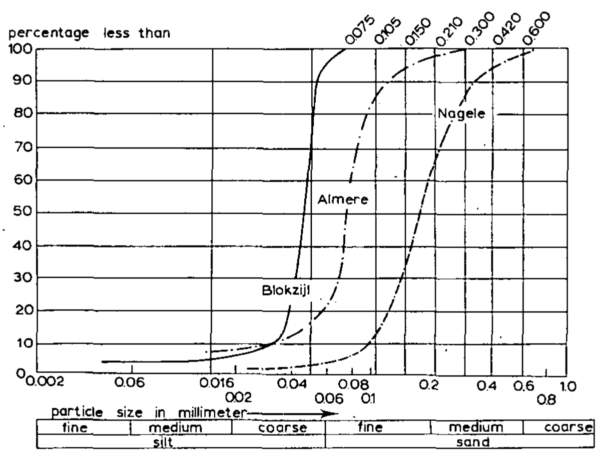

Fig.5. Particle size distribution curves for the sands used in the horizontal sand tank.

The discharge is measured together with the piezometric head inside the drain (h_o), just outside the drain plus envelope (h_1), and at 10 cm from the centre of the drain (h_2). In the interesting range of discharge rates, it appeared that a linear relationship existed between the discharge rate and the potential difference ($h_1 - h_o$ and $h_2 - h_o$).

$$W_e = \text{tg}\ \alpha = \frac{\Delta h(\text{m})}{q(\text{m}^3/\text{day}/\text{m})} \quad : \quad (\text{days/m}) \tag{4}$$

Some of the results obtained with three types of soil-fill are presented, for illustrative purposes only, in Table 2.

TABLE 2: Some results obtained with the horizontal sand-tank

TYPE OF ENVELOPE	Entrance resistance in days/metres determined in		
	Blokzijl sand	Almere sand	Nagele sand
Organic materials			
Fibrous peat	0.35	0.20	0.04
Flax straw	-	0.25	0.03
Oats straw	-	0.30	0.012
Coconut fibres	0.25	0.12	0.010
Synthetic materials			
Acrylic fibres	0.60	0.15	0.011
Fibre glass	-	0.20	0.014
Polyst. in PVC foil	0.34	-	-
Polyst. in PVA fleece	0.70	-	-
Polypropylene fibres	0.35	-	-
Without envelope	0.95	0.50	0.06

NOTE: Interpretation for very fine sands:

If:	$W_e < 0.25$	*Low*
	$0.25 < W_e < 0.50$	*Moderate*
	$0.50 < W_e$	*High*

(Data made available by courtesy of Ir.F.C. Zuidema of the IJsselmeer Polder Development Authority)

This type of sand-tank was chosen for the following reasons:

- Its set-up resembles field conditions
  - *a)* horizontal drain position with an equal soil pressure all along the drain;
  - *b)* soil types are typical soils found at drain depth in the newly reclaimed polders;
  - *c)* piezometer at 10 cm, being the outer boundary of a drain trench.
- Radial approach flow is approximated
- Consistent and comparable results are obtained by using homogeneous sands.

Two of the disadvantages of this set-up are that:

- Its preparation and operation is more time-consuming than for the vertical cylindrical model;
- Permeability of sand-fill cannot be easily derived, but has to be calculated presuming that full radial flow conditions prevail.

Field testing

Once new materials have shown promising results in the laboratory they have to be tested under field conditions. This is done in experimental fields. It is, however, quite an expensive activity and it takes up to three years before final conclusions can be drawn. Even longer periods of observations might be required to study long-term effects such as sedimentation, deposition of iron compounds, or age effects.

In the past 15 years quite a number of experimental fields have been set up by the Government Service for Land and Water Use (LD), and the IJsselmeer Polders Development Authority (RIJP). Both agencies have summarized their findings, but it is considered beyond the scope of this paper to review them.

It is, however, interesting to recall the main conclusions:

1. Coarse textured voluminous envelope materials such as fibrous peat and flax-straw perform their hydrologic function much better than more densely composed materials such as fibrous peat mixed with acrylic fibres, glass-wool, and fibre-glass fleece.
2. Differences in the performance of the various materials undergoing field tests are difficult to deduce from field measurements, because differences encountered very often have to be ascribed to inhomogeneities in the soil along one drain-line, differences between laterals in the same test plot, and variations between the different test plots.
3. The conditions at the time of installation have a much greater influence on the performance of the envelope materials than the minor differences between the types of materials.

One of the conclusions from the report of Miedema and Jonkers (1975), will have far-reaching consequences for the continuation and re-oriëntation of research on drainage materials:

> *"Taking into account the extremely high costs of the installation, follow-up, and execution of the measurement program of experimental fields and considering the limited value of the results, the question arises whether future research on drain envelopé materials ought to be conducted even more in the laboratory, testing only those materials under actual field conditions which have a fair chance of performing well".*

This conclusion calls for a thorough appraisal of the present testing procedure and should lead to suggestions for a new approach.

3. New research approach

3.1 Introduction

It is undeniable that in the future more emphasis will be placed on laboratory research. But, as we know, the greatest disadvantage of laboratory testing is the difficulty of transferring the results.

We need to arrive at a compromise between the aims of obtaining consistent and reproducible results and of testing new materials under conditions that closely resemble field conditions. We also need to be able to describe and quantify those physical properties of fibrous envelope materials that are of paramount importance for their performance in the field - i.e. their hydrologic and selective filtering function.

We must increase our knowledge of the interaction processes that occur between the surrounding soil and the envelope material under the influence of the groundwater flow towards and into the drain. The changes in a material's hydraulic conductivity as it becomes blocked or clogged by soil particle movement need more intense study.

Design criteria have to be developed for fibrous envelope materials (describing their thickness, pore size distribution and composition) as a function of the particle size distribution of the soil surrounding the drain. The key problem is the movement of soil particles under the influence of ground waterflow. On this subject two studies were conducted in 1976 at the Institute for Land and Water Management Research (ICW). They will be described below.

3.2 Theoretical study on the effect of envelope material on entrance resistance

The first study, conducted by Nieuwenhuis (1976), analysed the effect of perforation and envelope material on the entrance resistance and effective diameter of drain pipes. Some of the more important results are presented in the Figures 6 and 7.

Figure 6 presents the effect of varying envelope thicknesses on the entrance resistance and effective radius.

From this graph it can be concluded that:

- increasing the envelope thickness beyond approximately 10 mm will not further decrease the entrance resistance;
- the increase in effective radius with an increasing envelope thickness must be ascribed to a decreasing radial resistance.

  NOTE: The effective radius (Reff) is defined as the radius of an imaginary "ideal" drain offering the same flow resistance as the actual "non-ideal" drain.

  $$W_{tot} = W_{rad} \text{ "ideal" drain} = (W_{entr} + W_{rad}) \text{ "non-ideal" drain}$$

  Thus $Reff = R_s \exp(-2 \pi K W_{tot})$; *with* R_s *the radius of the considered flow region.*

- an increase in the ratio of $K_{envelope}$ over K_{soil} beyond 20 has little or no effect on a further decrease in entrance resistance.

It is interesting to note that a minimum thickness of 7 mm is prescribed in the standards for pre-wrappings of fibrous peat and coconut fibres, and that this decision was based on the results obtained in experimental fields and from practical experience.

Figure 7 presents the effect of a decrease in permeability over 20 per cent and 50 per cent of the outer perimeter of the envelope (clogging/blocking) on the entrance resistance and radial flow resistance for ratio of K_{env} over K_{soil} of 5.

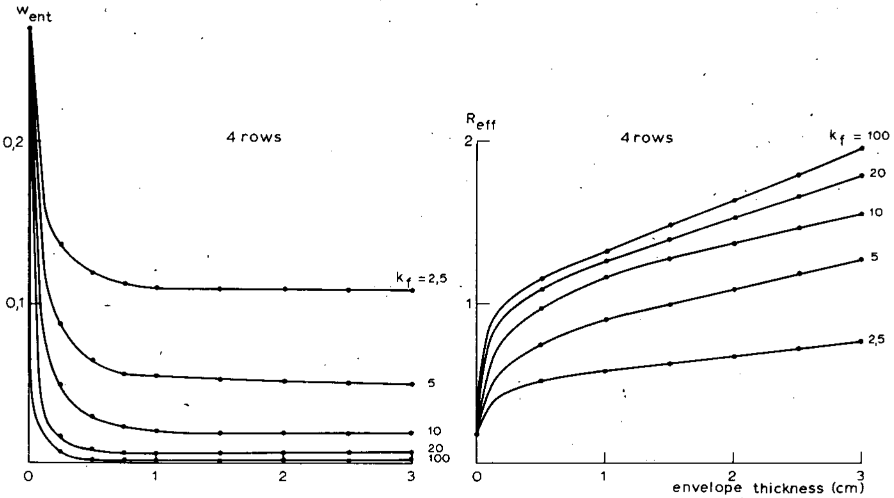

Fig.6. The influence of envelope thickness on the entrance resistance (W_e) and effective radius (Reff), theoretically analyzed for the two-dimensional case for a drain (ø 6 cm) with 4 rows of perforations (1 mm wide); (k_f = 100 means that the permeability of the envelope equals 100 times the permeability of the undistributed soil).

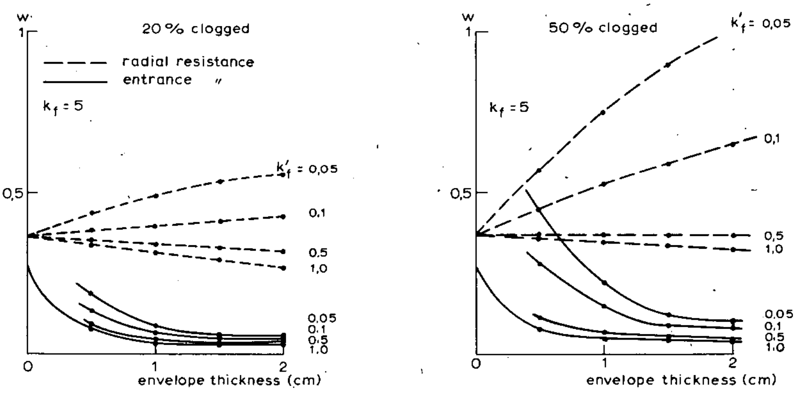

Fig.7. The influence of a partial clogging of the envelope by soil on the entrance and radial resistance. The permeability of the envelope (k_f) is taken as 5, while the clogged part is assumed to have a permeability (k'_f) of 1.0 to 0.05 times k_f.

From this graph it can be concluded that:

- clogging has a remarkably negative effect on performance, especially for the thin envelopes;
- if the clogged part of the envelope is 10 mm or more thick, the radial flow resistance increases considerably even though the entrance resistance may become negligible.

These results agree well with the users' preference for voluminous envelope materials.

3.3 Study of the flow processes in the immediate vicinity of the drain

The second study, conducted by Mr. N.I. Chrissanov[1], was performed in a small sand-tank, representing half the flow region. Under various degrees of drain "fill" and submergence, the flow processes and hydraulic gradients in the immediate vicinity of the drain were investigated. Two types of pipes (clay-tile with one joint, and corrugated PVC pipe), with and without envelope material were tested.

So far, the data collected have not been analysed, but observations indicate the existence of a seepage surface, the important hydrologic role played by a coarse structured envelope material and the occurrence of hydraulic gradients of more than one in the immediate vicinity of the drain.

3.4 Research approach to develop design criteria

Another study started in 1976, was the joint effort by ILRI and ICW mentioned in 2.1. Its objective is to obtain a better understanding of those physical characteristics of fibrous envelope materials that are crucial for their performance, and to develop design criteria for those characteristics as a function of the soil type.

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[1] *Scientific Research Worker of the North Research Institute, Hydrotechnic and Reclamation, Leningrad, USSR; guestworker in 1976 with the Institute for Land and Water Management Research, Wageningen.*

A tripartite approach is being followed:

Part 1: The determination of the pore-size distribution of fibrous envelope materials and the development of standard procedures and equipment for routine measurements of this distribution.

It is beyond all doubt that the pore-size distribution of the envelope is decisive for both the selective filtering and the hydrologic properties under given soil conditions.
The porosity distribution was derived from pF-curves and was based on the postulation that water is held in the pores of the synthetic or organic fibrous materials by capillary forces only.

Part 2: The investigation of the selective filtering properties in relation to pore-size distribution

This involved the measurements of changes in hydraulic conductivity caused by the blocking or clogging of the overlying soil fractions. The soil fractions used in the different testings corresponded with the equivalent pore sizes as determined in Part 1 of the study. The amount of soil passing through the envelope was also measured.

Part 3: The development of a quantitative description of interactions beween the fibrous envelopes and soil, in terms of ground waterflow and physical properties

So far only the first two parts of the study have been explored. It is our conviction, however, that when all three parts have been completed and their results are combined, a better understanding of envelope materials will be obtained.

3.5 Provisional results of the reconnoitring research

Determination of pore-size distribution

The measurement program, carried out by Eskes (1977), covered the pore-size distribution of various envelope materials under various conditions (i.e. different material thickness, effect of different soil loads, reproducibility of results).

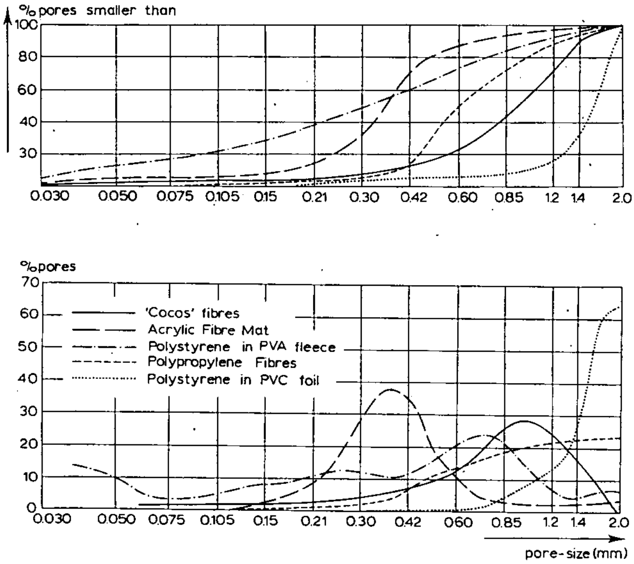

Fig. 8. Pore-size distribution of different envelope materials.

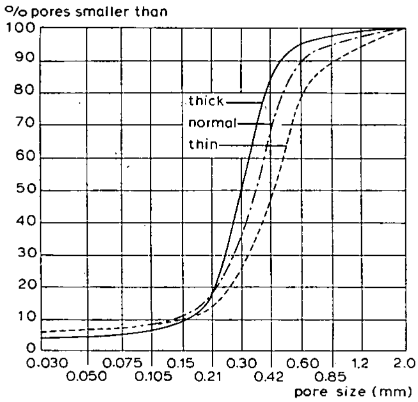

Fig. 9. The influence of the thickness on the pore-size distribution of acrylic fiber mat.

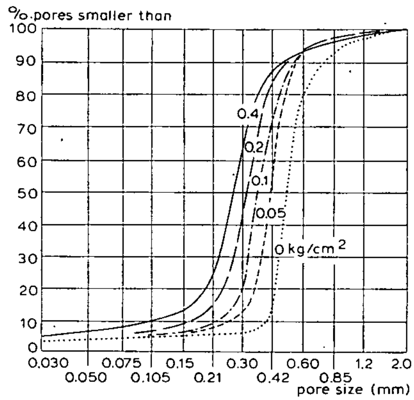

Fig. 10. The influence of different pressures on the pore-size distribution of acrylic fiber mat.

- Pore-size distribution of different materials.
  Figure 8 presents the results of the measurements on acrylic fibre mat, coconut fibre, cintamat (polystyrene flaxes binded by a polyvinyl alcohol fleece), polypropylene fibres, and polystyrene grains contained around the drain with a perforated PVC-foil.
- The relation between the porosity distribution and the thickness of the envelope was determined on acrylic fibre mat only.
  From each specimen, 5 samples selected at random were tested under simulated soil pressure of 0.2 kg/cm^2, roughly corresponding with a drain depth of 1 m.
  The results are presented in Fig.9 and summarized in Table 3.

TABLE 3. Tested samples of acrylic fibre mat

	Thickness mm	Material weight grams/m^2	70% of the pores in the range (mm)
thin	3	144	0.25 - 0.70
normal	5	223	0.20 - 0.50
thick	9	515	0.20 - 0.40

- The effect of different simulated soil pressures on the pore-size distribution was investigated for the "normal" specimen of acrylic fibre mat. Five samples selected at random were subjected to a load varying from 0.0 to 0.4 kg/cm^2. The average results for all five samples for each loadstep are presented in Figure 10.
  It is obvious that with an increasing load, the size of the pores decreases. For the non-loaded samples, 70% of the pores are between 0.4 and 0.6 mm, decreasing to between 0.2 and 0.4 mm under a pressure of 0.4 kg/cm^2. With increasing pressure, the variation in pore-sizes increased, which must be ascribed to deformations in the test samples as each sample was used for the full measuring programme.
- To verify whether consistent results could be obtained from the procedure followed, the same sample was analysed five times. The results are shown in Table 4.

TABLE 4. Percentage of pores obtained from repeated testings on the same sample (normal thickness and 0.2 kg/cm^2)

Range of pore-size (mm)	Test number 1	2	3	4	5
0.15 - 0.60	81	81	77	83	79
0.21 - 0.42	54	60	56	64	58

These data enable us to conclude that the methodology was sufficiently accurate to determine variations in the composition of the material and to produce workable and consistent results.

Investigations of selective filtering and hydrologic properties

The interaction between predetermined soil fractions and envelope materials under the influence of ground waterflow was studied in a vertical permeameter model (Eskes, 1977).

Only two envelope materials, acrylic fibre mat and coconut fibre, were investigated. They were provided with a corrugated "flattened" drain as base (a "flattened" drain is one which has been cut open and laid flat, leaving the corrugations unchanged). The materials were than subjected to a pressure simulating soil pressure. The following soil fractions were used for the test programme: 0.030-0.050; 0.050-0.075; 0.075-0.105; 0.105-0.210; 0.210-0.300; 0.300-0.420; 0.420-0.600; 0.600-0.850; 0.850-1.200; 1.200-1.410; 1.410-2.0 (all values in mm), and with Blokzijl sand (60% between 0.016-0.050 and 40% between 0.050-0.105).

The hydraulic gradients applied varied from 0.25 to 1 for the fractions from 2.0-0.6 mm and was 2 for all fractions less than 0.6 mm.

The following measurements were made:

- discharge measurements and piezometric readings;
- quantity of sediment passed through;
- quantity of sediment entrapped.

The quantity of soil particles passing through the envelope indicates its selective filtering properties, while the quantity entrapped indicates the hydrologic properties. The results are presented in Figures 11 and 12.

The following preliminary conclusions could be drawn on:

- *Quantity of soil particles passing through*

  Those soil fractions smaller than 1/2 to 1/3 the median pore size value of the envelope (ϕ_m) will start flowing through.

  Total throughfall (particles which pass entirely through the envelope) occurs for fractions smaller than 1/8 ϕ_m.

- *Quantity of soil particles entrapped*

  Particle entrapment in the envelope was observed for almost the entire range of tested particle sizes, with a peak between (1/4-1/8)ϕ_m. Similar results were obtained for Blokzijl sand on acrylic fibre mat.

- *Permeability of the envelope material*

  The particle sizes larger than 1/2 ϕ_m entrapped in the envelope negatively influence the permeability.

  For the soil fractions smaller than 1/2 ϕ_m, the permeability increases.

- *Effect of a sudden doubling of the hydraulic gradient*

  The permeability of soil fraction plus envelope decreases when the soil fractions have a similar size (dk) as the pores of the envelope material, with a maximal decrease for dk = ϕ_m.

  After a gradient doubling, exerted on Blokzijl sand in combination with acryl and cocos, the permeability stabilizes rapidly, with a 30 per cent decrease in permeability for acryl and 20 per cent for coconut fibres.

- *The boundary effect (blocking)*

  An increase in resistance (- sign in Figs.11 and 12) points towards a blocking effect in the boundary zone between soil and envelope. A decrease in resistance (+ sign in Figs.11 and 12) points towards the positive effect of a "natural filter" build-up in the boundary zone.

  Blocking generally occurs when the soil fraction (dk) is twice or more the median pore size (dk > 20 ϕ_m).

  For dk < 2 ϕ_m the blocking effect decreases with decreasing d_k.

  This might be explained by the development of a looser soil packing just above the envelope near the valleys of the corrugations, caused by the removal of particles and the elasticity of the envelope.

3.6 Conclusion

The results obtained so far are insufficient in number and repetitions to enable any significant conclusions to be drawn. The trends and relations observed between particle size of the soil, pore-size of the envelope, and the selective filtering and hydrological performance of envelope materials, have to be proved valid and reproducible on a much larger scale.

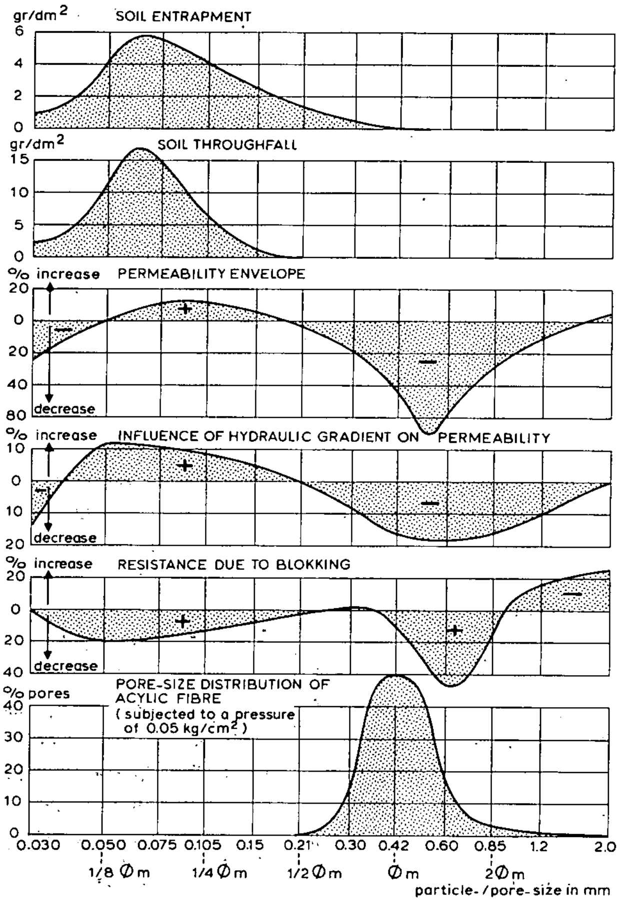

Fig. 11. *Acrylic fiber envelope and its functioning.*

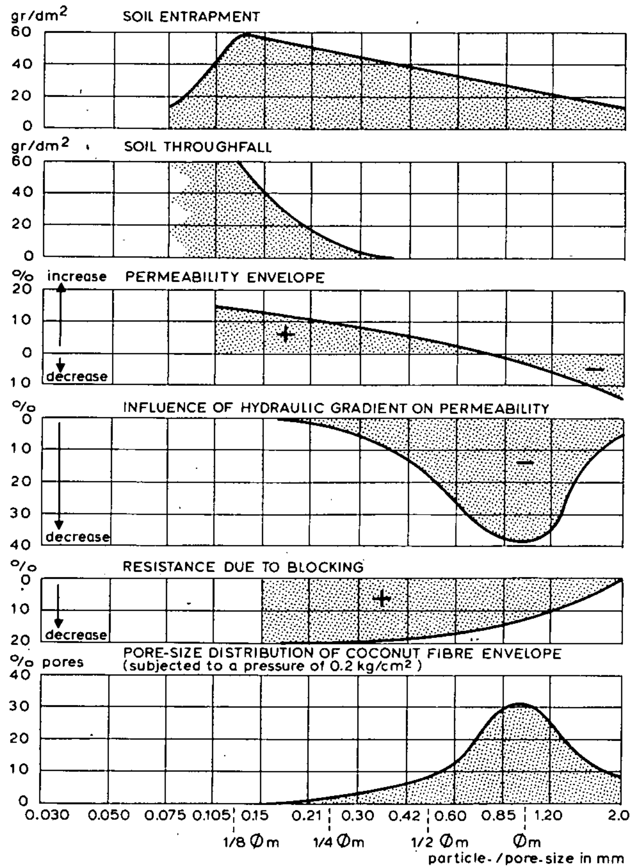

Fig. 12. *Coconut fiber envelope and its functioning.*

We can state, however, that these results were considered encouraging enough for us to continue the study along the lines proposed.

The major problem remaining is "How to bridge the gap between results obtained with soil fractions and actual problem soils?"

A first approach could be a gradual changeover from the analysis of soil fractions to standard soils. We will than have to combine the findings of this type of research with knowledge on the structural stability of soils, on soil mechanics, and on hydrodynamics.

It is beyond doubt that it will benefit the users as well as the manufacturers when we arrive at our pursued objective: the development of design criteria for fibrous drain envelopes. We believe that the first step has been taken towards this final goal by the research herein outlined.

To achieve our goal, however, the interest and cooperation of other scientists (e.g. in soil science and soil hydrodynamics) is indispensible.

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Paper 2.11

MODEL TESTS ON DRAINAGE MATERIALS

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Summary

The Scientific Department of the IJsselmeerpolders Development Authority tests new drain envelope materials on their practicality in sand tanks, followed by a final selection in experimental fields.

This article describes the testing arrangement in the laboratory, the measurement procedures, the evaluation techniques, and some results.

Introduction

In the sand tank of the Scientific Department of the Lake IJssel Polders Development Authority at Lelystad, new drainage materials are tested on their suitability for use in practice. Sometimes, for further selection of promising materials field experiments will follow. The laboratory experiments concern both pipe and filter materials. In this paper the test apparatus, the test procedure and some results are described.

The test apparatus

The test apparatus consists of an inner tank (130 cm long and 60 cm wide) filled with sand and placed in an outer tank to which water can be supplied and maintained at a certain level by means of a float and overflow (Fig.1). Via the perforated walls of the inner tank, water will penetrate into the sand and flow towards a horizontal length of drain in the lower part of the tank, one end having a free outflow.

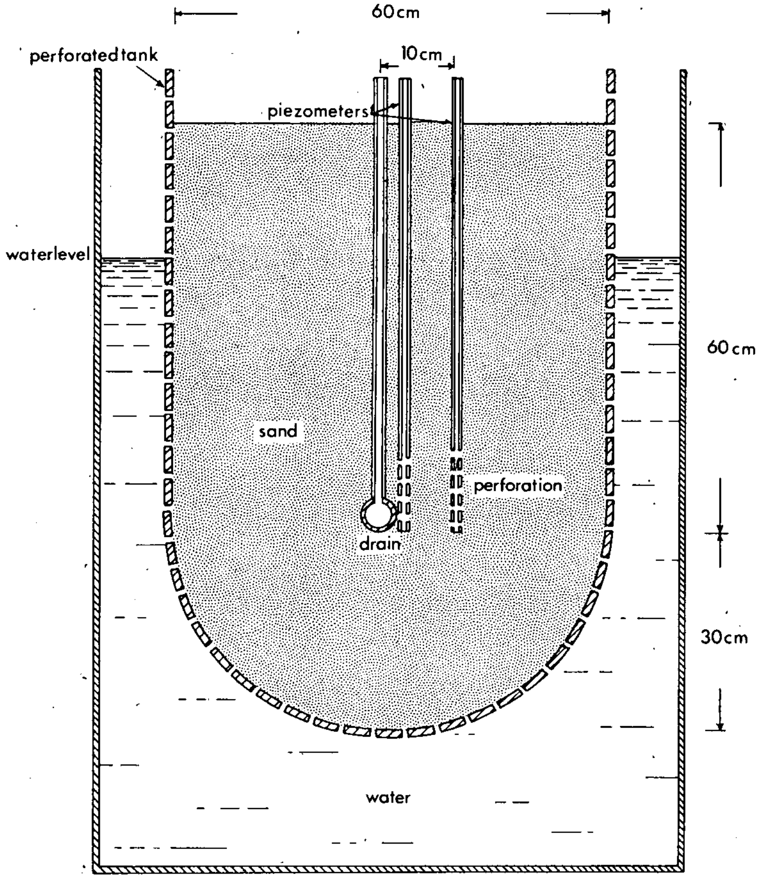

Fig.1. Cross-section of a drainage research tank.

Test runs are made with two types of sand fill in the tank viz. Blokzijl sand and Pleistocene sand, the first being a very fine sand, the second a middle fine sand. The grading of the materials is given in Table 1.

To prevent air entrapment in the sand when the tank is filled, the sand is brought in under water and regularly distributed. This helps to improve the consistency of the measuring results and enables the data from different materials to be compared.

Observations are also made of the washing-in of sand or other fine particles

TABLE 1. Grading of Blokzijl and Pleistocene sand

	nominal size (mm × 10^{-3})	<2	2–16	16–50	50–75	75–105	105–150	150–210	210–200	300–420	420–600	600–2000
Blokzijl sand	% by weight	3.4	2.0	52.5	31.4	3.1	0.1	0.1	0.1	0.1	0.1	0.1
Pleistocene sand	% by weight	1.0	1.0	4.3	7.3	11.5	28.2	28.7	12.3	3.2	1.0	1.0

through the filters into the drain pipe, by collecting, drying and weighing the sediment in the drain and outflow reservoir.

The test procedure

When the watertable in the outer tank is maintained at a higher level than the outflow of the drain, a curved watertable is developing between the wall of the inner tank and the drain which is measured by means of piezometers. At first the water in the outer tank is fixed at a certain level for three weeks, after the water supply is stopped, causing the level in the outer tank to lower gradually to drain level. During the whole period the drain discharge, the level in the outer tank and piezometric head at the outer side of the filter are simultaneously measured several times a day. From the potential difference Δh and the discharge q, the inflow- or entrance resistance W_e follows from $W_e = \Delta h/q$. The relation between the piezometric head and the drain discharge turns out for this test to be rectilinear. Therefore W_e can be characterized by the tangent of the angle between this line and the horizontal axis (tg α).

The results

As the behaviour of the watertable in the sand tank depends on the kind of drainage + filter material, the water level in the outer tank and the hydraulic conductivity of the used sand, the results of the measurements are fully comparable under the given conditions. They are presented as tg α-values in Table 2.

The influence of the type of sand can be derived by comparing the figures in these tables at the same combination of pipe and filter material. Figure 2 shows some results of tests on corrugated PVC pipes with various filter materials. The piezometric head is measured in two rows of piezometers placed, respectively just besides the drain and at 10 cm from the centre of the drain. So, from each test two lines are acquired, connecting the points that give the relation between the potential difference and the drain discharge.

The quantity of sand washed into the drain is given for each test in Table 2.

TABLE 2. Drainage tests in research tanks, filled with Blokzijl sand and Pleistocene sand. Entrance resistance of envelope materials, given as α-values, and the amount of washed-in soil particles in gram per meter drain

	Envelope material	Thickness envelope material (mm)	Blokzijl sand tg α-value a	b	Washing-in (g/m)	Pleistocene tg α-value a	b	Washing-in (g/m)
1.	coconut-fibre	1.7	0.25	1.20	975	0.055	0.140	0
2.	peat litter (envelope)	8.5	0.70	1.70	695	0.060	0.155	0
	peat litter (cover)	20-70*	0.15	0.50	2185			
3.	polypropylene fibre mat - 100 per cent rough fibres	5.5	0.30	1.30	2180	0.025	0.130	55
4.	polypropylene fibre mat 50 per cent rough 30 per cent middle 20 per cent fine fibres	4.9	0.45	1.35	940	0.035	0.145	<5
5.	polypropylene fibre mat mixture	6.5	0.15	1.65	865	0.025	0.135	<5
6.	polystyrene granules in perforated plastic sheet:							
	perf. spacing 5 mm	9.5	0.30	1.30	1340	0.020	0.135	280
7.	perf. spacing 7 mm	13.0				0.025	0.100	220
8.	perf. spacing 10 mm	10.6	0.35	1.40	2215	0.040	0.125	160
9.	polystyrene flaxes in polyvinyl alcohol fibre membrane	9.6	0.70	1.50	0			
10.	acryllic fibre mats	4.0	0.60	1.45	0	0.045	0.145	0
11.	polypropylene fibre mats	2.1	0.95	1.85	<5	0.040	0.140	0
12.	polypropylene envelope	1.0	0.65	1.70	340			
13.	glass-fibre sheet	0.1	0.65	1.35	0	0.045	0.175	0
14.	without envelope	0	1.45	2.45	1900	0.080	0.205	810

a = piezometers just aside the drain
b = piezometers 10 cm heart of the drain

**20 mm above the drain and 70 mm aside the drain*

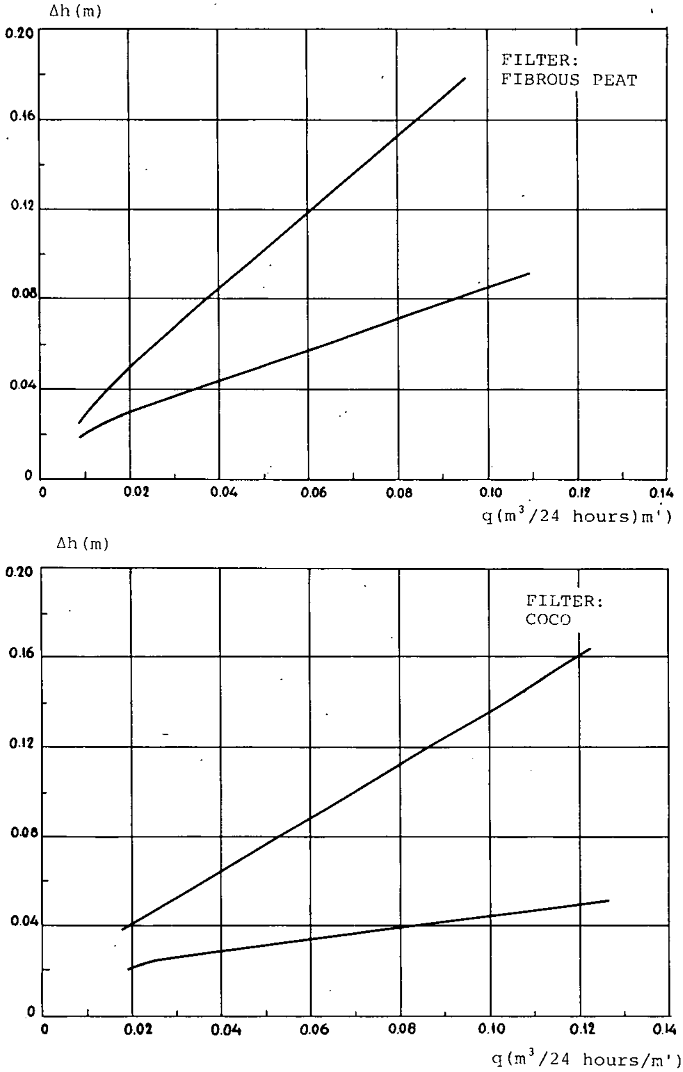

Fig. 2. Relation between potential difference Δh and drain discharge q.

Measurements on corrugated P.V.C. pipes in Blokzijl sand.

Tested materials

The materials tested in the sand tanks can be divided into two groups:

1. *Organic materials*. During the last few years these voluminous envelope materials (coconut fibre and peat litter) have been applied in the field

2. *Synthetic materials*. This concerns voluminous materials as well as thin fibrous or non-woven membranes. Up to this moment only thin acryllic fibre mats (thickness 3 mm) and glass fibre have been applied on a larger scale than in field tests.

From the data of Table 2 the following may be concluded:

1. Generally speaking, in very fine sand (Blokzijlsand) the entrance of the voluminous envelope materials is considerably lower than that of thin fibrous envelopes. There are two exceptions: polystyrene granules in sheeting, where the perforated plastic sheeting is the bottle-neck, and peat litter which becomes very dense when applied as envelope material. If the peat litter was used as loose material on and beside the pipe, the entrance resistance was much lower, because of the presence of a layer of peat of about 7 mm thickness beside the drain pipe.

The hydralic conductivity of Blokzijlsand used in the tank, is 0.15 m/day. In practice a soil with such (low) K-value will be drained by pipes with a spacing of 8 m. At this drain spacing and under discharge conditions of 0.01 m/day a hydraulic head of 8 cm between the water level in the drain and just aside the drain means a tg α-value of 1.0. Under these conditions the extra hydraulic head midway between two drains will be about 50 cm.

Such a situation will lead to the loss of structure and increasing compaction of the trench backfill and may influence cultivation operations in a negative way. Therefore a tg α-value of 0.5 is considered as maximum acceptable figure

2. The tg α-values of the materials, tested in the middle fine sand do not differ very much. It is remarkable that the organic materials coconut

fibre and peat litter show the highest tg α-values. However, the differences are small when transferred into practice. The hydraulic conductivity of the Pleistocene sand in the sand tank K=1.5 m/day. A soil with such a K-value will be drained at spacings of 25 m. A drain spacing of 25 m, combined with a design discharge of 0.01 m/day means a tg α-value of 0.05 and a hydraulic head of 1,2 cm.

3. Thin woven and non-woven envelope materials show a good filtration function in very fine sand, in contrast with the voluminous materials polypropylene fibre mats and polystyrene granules in a perforated plastic sheeting. This follows from a comparison of the quantity of sand washed-in in the drain pipes with thin, voluminous and without an envelope.

In middle fine sand the filtration function of nearly all tested materials is good, except the polystyrene granules in perforated plastic sheeting.

The composition of the washed-in material has been analysed for some tests in Blokzijl sand and Pleistocene sand (see Tables 3 and 4).

TABLE 3. Drainage tests in Blokzijl sand. Composition of the washed-in material in comparison with the original soil

Envelope	% of the dry soil		sub-fractions of the sand in % ($mm \times 10^{-3}$)							
	lutum	sand	16-50	50-75	75-105	105-150	150-210	210-300	300-420	420-600
polypropylene fibre mat										
100% rough	3.0	87.5	64.8	20.8	1.2	0.4	0.1	0.1	0	0
50% rough, 30% middle, 20% fine	2.3	87.6	63.5	21.8	1.4	0.5	0.2	0.1	0.1	0
polystyrene granules in perforated plastic sheet	2.9	88.8	60.6	23.9	3.3	0.6	0.2	0.1	0.1	0
polypropylene sheet	3.5	86.4	62.6	21.9	1.3	0.3	0.2	0.1	0.1	0
coconut fibre	3.5	86.2	63.3	20.6	1.2	0.4	0.4	0.2	0.1	0
without envelope	2.0	88.8	64.2	22.1	1.4	0.4	0.3	0.2	0.1	0.1
sand in research tank	3.4	88.0	52.5	31.4	3.1	0.5	0.1	0.1	0.1	0.1

Table 3 shows that the percentage of the sub-fraction 16-50 micron of the washed-in sand is considerably higher than of the original material in the tank. It is remarkable that the use of a filter material does not change this situation, as may be derived from the figures concerning the drain without envelopes.

The washed-in material does not differ much from the original sand composition, except for the envelope with the biggest spacing of the perforations. Here a relatively larger amount of coarser particles has moved into the drain pipe, probably due to a higher flow velocity in the surroundings of the perforations.

4. A decrease of the amount of perforations in the plastic sheet that keeps the polystyrene granules together, will increase the entrance resistance. However, this value is still acceptable for the sheet with the smallest number of perforations (spacings 10 mm).

The tests in pleistocene sand do not result in relations between tg α-value and the number of perforations. This can be explained from the various thicknesses of the envelope materials. In the same sand the weight of the washed-in material is higher when the amount of perforations per meter is lower. This cannot be concluded from the tests in Blokzijl sand.

TABLE 4. Drainage tests in Pleistocene sand. Composition of the washed-in material in comparison with the original soil

envelope	% of the dry soil		sub-fractions of the sand in % ($mm \times 10^{-3}$)							
	lutum	sand	16-50	50-75	75-105	105-150	150-210	210-300	300-420	420-600
polystyrene granules in plastic sheet										
spacing of perforations:										
10 mm	2.4	95.8	1.7	3.6	7.5	25.7	33.2	17.0	5.5	1.3
7 mm	1.6	97.4	3.0	6.6	11.9	31.8	27.8	12.1	3.4	0.8
5 mm	1.8	97.9	2.5	5.7	11.0	32.3	30.9	11.7	3.1	0.6
without envelope	1.0	97.5	4.1	6.9	11.0	28.1	29.3	13.0	3.3	1.0
sand in research tank	0.5	97.7	4.3	7.3	11.5	28.2	28.7	12.3	3.2	1.0

Conclusions

Tests of drainage materials in two sand tanks, filled with very fine sand, respectively middle fine sand lead to the following conclusions:

1. Thin, woven or non-woven envelope materials are not advised for use in very fine sandy soils. These materials will prevent washing-in of soil particles (filtration function), but cause a high entrance resistance. The same holds for polystyrene flaxes in a polyvinyl alcohol membrane and for the peat litter envelope material.

2. In middle fine (and coarse) sandy soils; woven and non-woven envelope sheets may be applied, if there is chance on iron-, sulphite-, or sulphate deposits in the pipe.

3. The entrance resistance of two new synthetic, voluminous envelope materials, namely polypropylene fibre mat and plastic sheet, is at an acceptable low level. If polypropylene fibre mat is applied in fine sandy soils, washing-in of soil particles can be prevented by using a thicker fibre mat with a mixture of rough and fine fibres. The percentage of fine fibres is limited, as a high amount will influence the entrance resistance of the envelope in a negative way. Recently a ratio of 70 per cent rough fibres (200 denier) and 30 per cent fine fibres (70-90 denier) has been advised for practical use.

The washing-in of soil particles at polypropylene granules can be diminished by the application of a smaller granule diameter.

Paper 2.12

COMPOSING A DRAINAGE PIPE LINE OUT OF SECTIONS WITH DIFFERENT DIAMETERS

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Summary

Formulas and graphs for hydraulic design of drain pipe lines, assuming varied flow and full-flowing pipes, are only valid if the entire pipe line consists of one and the same diameter. If a drain is to be composed of sections with different pipe diameters, then a correct design can be obtained by using each diameter till a certain percentage only of the drainable area (or drain length) as derived from the formulas and graphs. Percentages to be taken for practical design purposes are: 85% in case of 2 different diameters; and 75% in case of 3 or more different diameters.

1. Introduction

The flow equations used for hydraulic design of pipe drains may be based on either one of two different concepts, viz:

1. *Uniform flow*, assuming a constant flow rate over the entire length of pipe line considered. It implies that the maximum flow rate (drainage coefficient multiplied by area drained) which, in fact, occurs only near the outlet of a pipe drain, is assumed to apply to the entire pipe line. This concept is still widely used internationally in drainage design practice.

2. *Non-uniform flow* or *varied flow*, assuming a gradually increasing flow rate from zero at the upstream end till a maximum at the outlet; as the drain is assumed to take up water uniformly over its entire length. This concept is normally used in The Netherlands.

The varied flow-equations give an accurate description of the maximum flow capacity of full-flowing drain pipes. They are, however, only valid if the entire drain line consists of the same diameter. If a drain line is to be composed of sections with increasing diameters in the flow direction, some adaptations are necessary, which will be discussed in this paper.

2. General formulation of flow through drain pipes

The general equation for full-flowing pipes can be written as:

$$i = \frac{z}{x} = a\ d^{-\alpha}\ Q^{\beta} \tag{1}$$

or

$$Q = a^{-\frac{1}{\beta}}\ d^{\frac{\alpha}{\beta}}\ i^{\frac{1}{\beta}} \tag{1a}$$

in which

- i = hydraulic gradient
- z = loss in hydraulic head
- x = length of pipe line section considered
- d = pipe diameter
- Q = rate of flow
- a, α and β are constants, depending on the type of flow, wall roughness, etc.

In case of land drainage, the flow rate increases gradually from zero at the upstream end to a maximum at the outflow; consequently in the case of *full-flowing pipes* (which will be considered henceforth) also the hydraulic gradient increases in the direction of flow (see Fig.1). At a distance x from the upstream end:

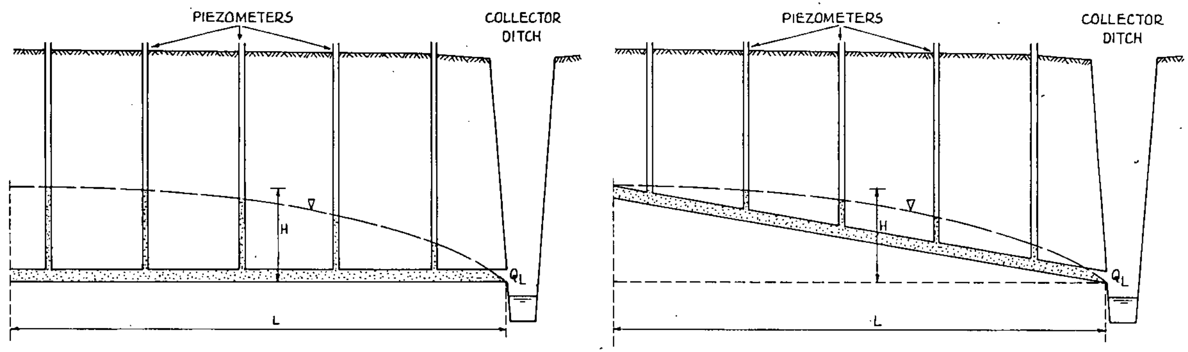
$$Q_x = q\ B\ x$$

in which

- q = drainage coefficient (depth of water to be removed per unit of time)
- B = width of area to be drained, measured perpendicularly to the direction of drain (which is, as a rule, equal to the drain spacing).

Applying Eq. 1 yields (see also Fig.2):

$$i = \frac{dz}{dx} = a\ d^{-\alpha}\ (qB)^{\beta}\ x^{\beta} \tag{2}$$



a. HORIZONTAL DRAIN

b. DRAIN HAS A SLOPE EQUAL TO THE AVERAGE HYDRAULIC GRADIENT

FIG. 1 HYDRAULIC GRADIENTS (PIEZOMETRIC CURVES) IN FULL-FLOWING DRAIN PIPE LINES (VARIED FLOW)

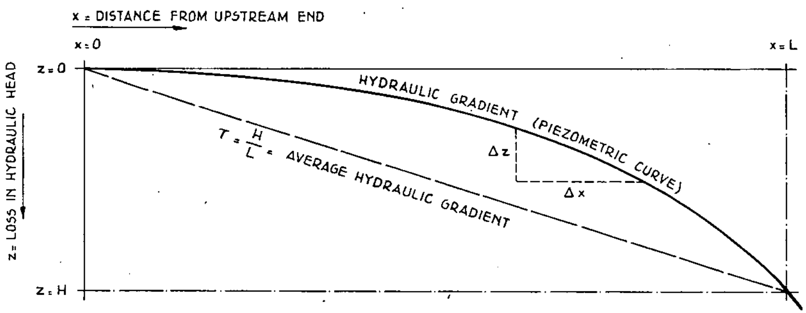


FIG. 2 HEAD LOSS IN A FULL-FLOWING DRAIN PIPE (VARIED FLOW) AS A FUNCTION OF DISTANCE

After integration

$$z = \frac{1}{\beta+1}\, a\, d^{-\alpha}\, (qB)^{\beta}\, x^{\beta+1} + C \tag{3}$$

or

$$z = a'\, d^{-\alpha} (qB)^{\beta}\, x^{\beta+1} + C \tag{3a}$$

if $$a' = \frac{1}{\beta+1}\, a \tag{3b}$$

For a drain line, consisting of one diameter only, the following conditions apply:

$$z = o \text{ for } x = o$$

$$z = H \text{ for } x = L$$

$$H = a'\, d^{-\alpha}\, (qB)^{\beta}\, L^{\beta+1} \tag{4}$$

$$\bar{i} = \frac{H}{L} = a'\, d^{-\alpha}\, (qBL)^{\beta} \tag{5}$$

$$= a'\, d^{-\alpha}\, Q^{\beta} \tag{5a}$$

and $$Q = qBL = a'^{-\frac{1}{\beta}}\, d^{\frac{\alpha}{\beta}}\, \bar{i}^{\frac{1}{\beta}} \tag{5b}$$

In these equations, H is the total head loss over a distance L and $\bar{i}$ is the *average hydraulic gradient*. This average gradient is usually one of the conditions in hydraulic calculations.

Quite commonly, the slope of the drain line is specified which implies the condition that the average hydraulic gradient should not exceed the slope of the pipe line or, that there should be no overpressure at the upstream end of the drain line (although there may be some overpressure between upstream end and outflow; see also Fig.1b).

Given pipe diameter and slope (maximum average gradient), the maximum drain length follows from:

$$L = a'^{-\frac{1}{\beta}}\, d^{\frac{\alpha}{\beta}}\, (qB)^{-1}\, \bar{i}^{\frac{1}{\beta}} \tag{6}$$

Considering different pipe diameters d_1 and d_2, then their maximum permissible drain lengths are related according to:

$$\frac{L_2}{L_1} = \left(\frac{d_2}{d_1}\right)^{\frac{\alpha}{\beta}} \qquad (7)$$

or:

$$L_2 = L_1 \left(\frac{d_2}{d_1}\right)^{\frac{\alpha}{\beta}} \qquad (7a)$$

3. Increasing pipe diameters

Figure 3 shows hydraulic gradients for different pipe diameters, together with an assumed slope or average gradient which is not to be exceeded. Then the maximum length

for $d_1 = L_1$
for $d_2 = L_2$, etc.

However, this holds only if the entire drain line consists of one and the same diameter.

If a drain is composed of increasing pipe diameters in the flow direction, then the head loss is found by fitting the gradients together as is done in Fig. 4. From this figure it also becomes immediately clear that it is *not correct* to use diameter d_1 for a length L_1, then d_2 from L_1 to L_2, etc. The head loss becomes too much and the hydraulic gradient is considerably more than the slope that was taken as a standard.

A correct method of composing a drain line out of increasing pipe diameters can be obtained by fitting gradients together in such a way that the hydraulic gradient does not cross the assumed slope (see Fig.5). This can conveniently be carried out by means of transparants, on each of which the gradient for one diameter has been drawn. At the interchange from one diameter to the next, the gradients are connected by shifting in a vertical direction over a certain distance.

In this way there is an infinite number of solutions possible; in order to find a specific solution, one more condition needs to be introduced. Van der

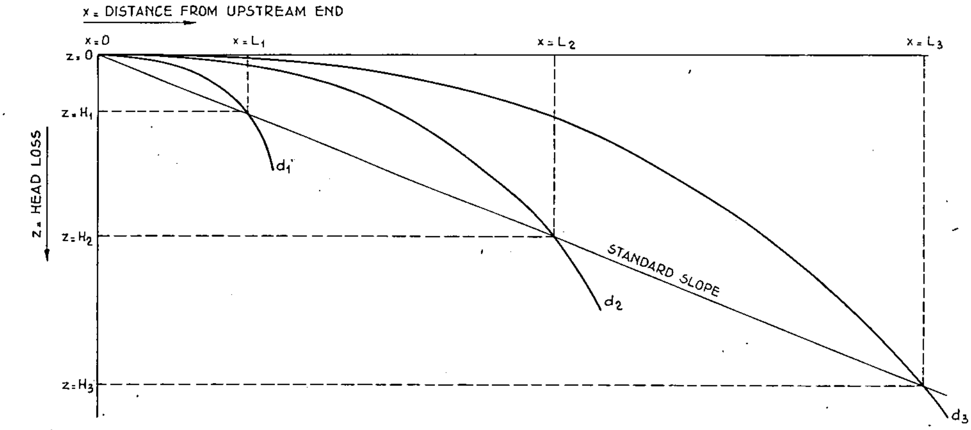


FIG. 3 HYDRAULIC GRADIENTS (PIEZOMETRIC CURVES) FOR DIFFERENT DIAMETERS

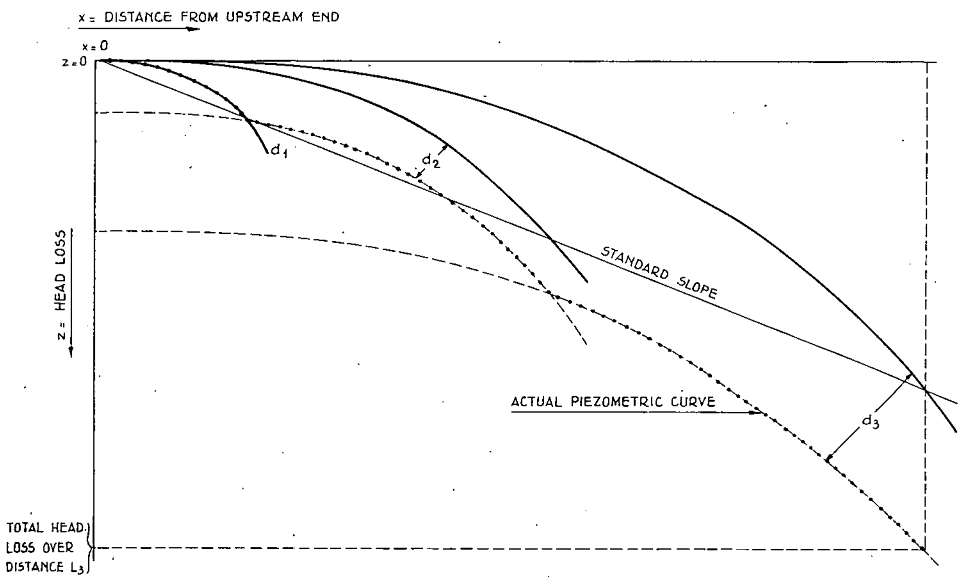


FIG. 4 HYDRAULIC GRADIENT (PIEZOMETRIC CURVE) OF DRAIN, CONSISTING OF INCREASING PIPE DIAMETERS

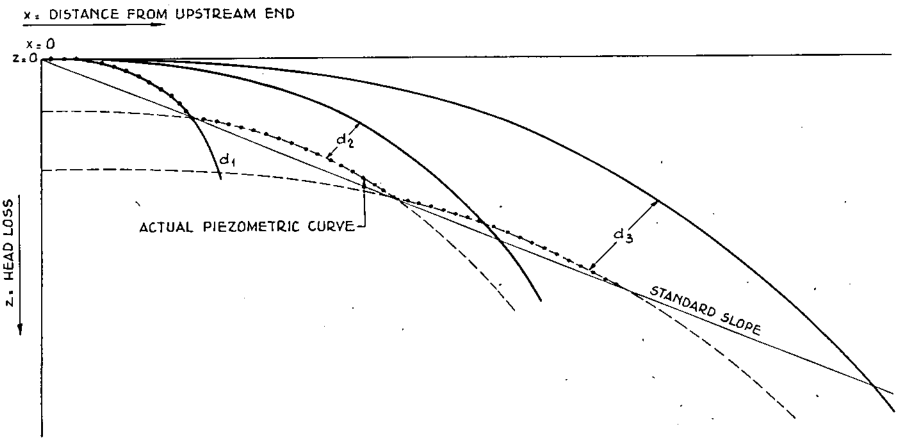


FIG. 5 HYDRAULIC GRADIENT OF DRAIN WITH INCREASING PIPE DIAMETERS; AVERAGE GRADIENT DOES NOT EXCEED STANDARD SLOPE

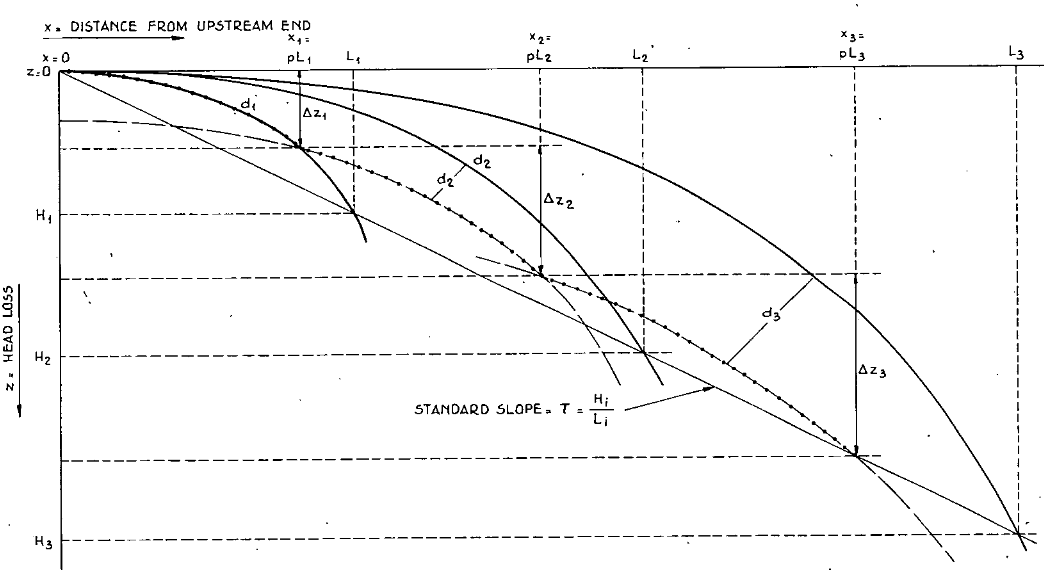


FIG. 6 HYDRAULIC GRADIENT OF DRAIN WITH INCREASING PIPE DIAMETERS; MAXIMUM DISTANCE FOR EACH DIAMETER REDUCED TILL CONSTANT PERCENTAGE = 100 p

Molen (1960) set the condition, that *in each section* the average gradient should be equal to the assumed slope. This implies, that the smallest diameter is used till its maximum permissible length (see example in Fig.5).

A disadvantage is that comparatively much head is lost in the last portion of the smallest diameter. It is then necessary to shift to a considerably larger diameter, otherwise the gradient would immediately come below the "standard slope". As an example: by this method it is not possible to have a 100 mm pipe followed by a 125 mm pipe (or 200 mm by 250 mm).

3.1 New method

According to the method to be discussed here, the shift from any diameter to the next one is done at a certain percentage of the maximum permissible length valid for that particular diameter; in other words:

Consider a drain line, consisting of <u>n</u> sections of increasing diameters d_1, d_2 ... d_n, then the shift from d_i to d_{i+1} is at a distance $x_i = pL_i$, where p is equal for each diameter and should be such that over the entire drain line the average hydraulic gradient should not exceed the standard slope (see Fig.6),

or:

$$\frac{z_t}{x_t} = \frac{z_t}{pL_n} \leqq \frac{H_n}{L_n} \tag{8}$$

$$\frac{z_t}{p} \leqq H_n \tag{8a}$$

in which

z_t = total head loss over entire drain line
x_t = total length of entire drain line
$\frac{H_n}{L_n}$ = the standard slope $\bar{i}$ for diameter d_n.

Now consider section No. i with diameter d_i, then, according to Eq.3a

$$\Delta z_i = a' d_i^{-\alpha} (qB)^{\beta} (x_i^{\beta+1} - x_{i-1}^{\beta+1}) \tag{9}$$

in which

x_i = the distance (measured from the upstream end of the drain line) till where diameter d_i has been used

x_{i-1} = as x_i, but for diameter d_{i-1}

So, the length of section i = $x_i - x_{i-1}$

Further we had

$$x_i = pL_i \tag{10}$$

Substitution of (10) in (7a) and (9) yields:

$$\Delta z_i = a'p^{\beta+1} L_n^{\beta+1} d_n^{-\alpha} \left[\left(\frac{d_i}{d_n}\right)^{\frac{\alpha}{\beta}} - \left(\frac{d_{i-1}}{d_i}\right)^{\alpha} \left(\frac{d_{i-1}}{d_n}\right)^{\frac{\alpha}{\beta}} \right] \tag{11}$$

$$= a'p^{\beta+1} L_n^{\beta+1} d_n^{-\alpha} T_i \tag{11a}$$

$$z_t = \sum_{i=1}^{n} \Delta z_i = a'p^{\beta+1} L_n^{\beta+1} d_n^{-\alpha} \sum_{i=1}^{n} T_i \tag{12}$$

We further introduce:

$$\sum_{i=1}^{n} T_i = T \tag{13}$$

Substitution of (13), (12), and (4) in (8) yields:

$$p^{\beta} T \leqq 1 \quad \text{or} \quad p \leqq T^{\frac{1}{-\beta}} \tag{14}$$

Elaboration of T yields:

$$T = \left(\frac{d_1}{d_n}\right)^{\frac{\alpha}{\beta}} \left[1 - \left(\frac{d_1}{d_2}\right)^{\alpha} \right] + \left(\frac{d_2}{d_n}\right)^{\frac{\alpha}{\beta}} \left[1 - \left(\frac{d_2}{d_3}\right)^{\alpha} \right] +$$

$$+ \text{- - - - - - - -} + \left(\frac{d_{n-1}}{d_n}\right)^{\frac{\alpha}{\beta}} \left[1 - \left(\frac{d_{n-1}}{d_n}\right)^{\alpha} \right] + 1 \tag{15}$$

TABLE 1. Values of p (reduction of maximum length of drain) for difficult combinations of diameters, both for 'hydraulic smooth' and 'hydraulic rough' flow

n = number of sections	Combination of diameters (proportional figures)						Value of p: for hydraulic smooth $\alpha = 4.75$, $\beta = 1.75$	Value of p: for hydraulic rough $\alpha = 5.333$, $\beta = 2$
2					8	10	0.840	0.850
3				6	8	10	0.781	0.794
4			5	6	8	10	0.756	0.771
5		4	5	6	8	10	0.742	0.757
6	3	4	5	6	8	10	0.735	0.751
2					7.5	10	0.864	0.856
3				6	7.5	10	0.792	0.805
4			4	6	7.5	10	0.771	0.786
4			5	6	7.5	10	0.766	0.781
5		4	5	6	7.5	10	0.752	0.767
2					9	10	0.862	0.869
3				8	9	10	0.784	0.795
4			7	8	9	10	0.736	0.750
5		6	7	8	9	10	0.706	0.721
2					6	10	0.889	0.898
3				5	6	10	0.855	0.865
4			4	5	6	10	0.835	0.846
3				4	6	10	0.861	0.872
3				6	9	10	0.790	0.803
4			5	6	9	10	0.765	0.779

3.2 Results and discussion

Table 1 shows the value of p for a numder of combinations of diameters.These diameters are to be seen as proportional sizes. Example: the combination 6-8-10 may stand for 60-80-100 mm or 120-160-200 mm or 300-400-500 mm.

It is seen that p depends on:

a) The number of sections: p decreases as the number of sections increases.
b) The interval between successive diameters: intervals.
c) The type of flow (hydraulic smooth or hydraulic rough).

It appears that the number of sections is the main deciding factor. For practical purposes the following general rule can safely be applied:

- if a drain line is to be composed of 2 different diameters, then the maximum permissible length for each diameter is 0.85 that of the maximum length if only one diameter were used;

- in case of 3 or more different diameters the maximum length should be reduced to 0.75.

There are only a few cases where p is considerably lower than 0.75, but these are combinations of diameters that are most unlikely to be applied in practic

3.3 Example

Consider a collector, consisting of cement pipes, that is to meet the following design specifications:

- slope of pipe line (average hydraulic gradient) = 0.05%
- drainage coefficient q = 7 mm per day
- width of area to be drained (B) = 300 m
- to allow for a safety margin, use only 75% of theoretical drainable area.

The following equation is used for the design:

$$Q = qBL = 89\ d^{2.714}\ \bar{i}^{0.572}$$

valid for varied flow, assuming full-flowing pipes.

In case only one pipe diameter is used for the entire collector, then the maximum drainable areas and maximum collector lengths for some diameters are as given in the table below:

diameter (mm)	150	200	250
Q (m^3 per day) 100%	580	1260	2300
Q (75% to allow for safety margin)	435	945	1725
L (75% to allow for safety margin)	210	450	820

If the collector is to be composed of 200 and 250 mm pipes, then the maximum lengths for each diameter will be as follows:

- 200 mm: up to 385 m (= 85% of 450 m) from the upstream end
- 250 mm: between 385 m and 700 m (=85% of 820 m)

In case of 3 different diameters, viz 150; 200 and 250 mm respectively, then the maximum lengths become:

- 150 mm: up to 155 m (=75% of 210 m)

- 200 mm: from 155 m till 340 m (=75% of 450 m)
- 250 mm: from 340 m till 620 m (=75% of 820 m)

References

VAN DER MOLEN, W.H. 1960. Over de diameter van drains en hoofddrains. (On the diameter of lateral and collector pipe drains.) Rep.Grontmij N.V.De Bilt, The Netherlands.

Paper 2.13

THE DISCHARGE CAPACITY OF CORRUGATED PLASTIC DRAIN PIPES, DETERMINED BY FIELD DATA

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Summary

Analyses of field data on discharge and hydraulic heads from corrugated pipes indicate that a hydraulic rough turbulent flow in the pipe allows a good explanation for the measured hydraulic head losses. The roughness coefficient apparently is related to the Reynolds number $\lambda = 7.0\ Re^{-0.5}$ for clean and well-maintained pipes.Field data indicated that for badly maintained and partly silted drainlines the coefficient 7.0 may increase to about 10 to 14. On the basis of the derived flow equation this implies that a silting percentage of 22% to 30% can be responsible for this increase. The latter figure can be explained when a silt layer of 13 mm is present on the bottom side of the line. Allowing the same thickness of a silt layer in larger diameter pipes, a reduction in drainable area is found which is dependent on the applied pipe diameter. The data introduced in this way have been worked out in a nomograph that can be used for design purposes.

1. Introduction

In drainage design the diameter of the pipe must be such that the discharge capacity of the system is sufficiently large to transport the drainage water without difficulties. In The Netherlands a nomograph constructed by IJssel Lake Polder Authority is used to determine the proper drain pipe diameter.

The nomograph relates discharge coefficient, hydraulic gradient and diameter so that for every chosen diameter of pipe the permissible drainable area can be derived.

Zuidema and Scholten (1971) give a full description of the way the nomograph has been derived. The basic principle of the nomograph is that the Darcy-Weisbach roughness coefficient λ depends on the Reynolds number R_e according to the general equation

$$\lambda = a\, R_e^{-b} \tag{1}$$

On the basis of laboratory experiments of Wesseling and Homma (1967) the value of b has been chosen to be 0.25, a value for a hydraulically smooth type of flow.

Applying the drainage principle, by the assumptions that the flow into the drain line is constant all over its length, the hydraulic head h_x required over a distance x of a drain line of total length L (meters) is (Wesseling 1965).

$$h_x = 9.55 \cdot 10^{-4}\, a\, D^{-4.75} \left(\frac{q}{L}\right)^{1.75} \{L^{2.75} - (L-x)^{2.75}\} \tag{2}$$

where

D = inner diameter of drain pipe in m

q = discharge from the outlet of the pipe in m^3/s.

Zuidema and Scholten now determined the a-value of different types of drain pipes under field conditions by plotting field data on hydraulic head h_x at different distances x from the outlet against the discharge q. They arrived at a-values for smooth plastic and clay pipes of 0.41. For corrugated PVC-pipes they found a-values of 0.78. On the basis of these data the nomograph was constructed.

Laboratory measurements on corrugated pipes show that the flow in this type of pipes may not be characterized as hydraulic smooth. This was the reason to reconsider the hydraulic heads acquiring in corrugated pipes under field conditions.

For this purpose field experimental data collected by the Government Service for Land and Water Use in the Province of Zeeland were used. On the basis of the results a new nomograph for the determination of the pipe diameter was constructed.

2. Present approach to the discharge capacity of drains

Accepting the drainage principle, that is a constant inflow per unit length of drain pipe, the flow inside the drain may be expected to change gradually from a laminair type at the upper end through a transition zone to a turbulent type of flow closer to the outlet end.

Laboratory experiments indicate that corrugated pipes have hydraulically rough properties. Despite that, it is assumed that at least for the higher Reynolds numbers the relation between roughness coëfficient and Reynolds number is represented by Eq.1.

Using this relation the head loss in a drain pipe can be described as (Wesseling 1965)

$$h_x = \frac{a}{3-b} \frac{\nu^b}{2g} \left(\frac{4}{\pi}\right)^{2-b} \left(\frac{q}{L}\right)^{2-b} \left[L^{3-b} - (L-x)^{3-b}\right] D^{b-5} \qquad (3)$$

where ν is the kinematic viscosity and g the acceleration of gravity.

Data from different types of drains in East Flevoland (Fig.1) showed that b=0.5 and a-values independent of the discharge. Field data from experimental fields in Zeeland with corrugated pipes (D = 0.044 m inside) showed that at least for q > 0.2 l/s a value b = 0.5 is reasonable (Fig.2). The lower discharges give higher values of b. Apparently a larger part of the drain pipe is subject to laminar flow. Also the uncertainty of lower hydraulic measurements may have some influence.

For design purposes, however, these flow quantities are of little or no importance.

The consequence of this fact is that drain discharge capacities can be based on the equation

$$h_L = 3.36 \times 10^{-5}\ a\ \frac{q^{1.5}}{D^{4.5}}\ L$$

where q is the total amount to be discharged by a single drain, L the length of the drain line and h_L the available hydraulic head.

FIG 1 Relation between hydraulic head and draindischarge

Approaching b-value by $h_x = A . q^{2-b}$

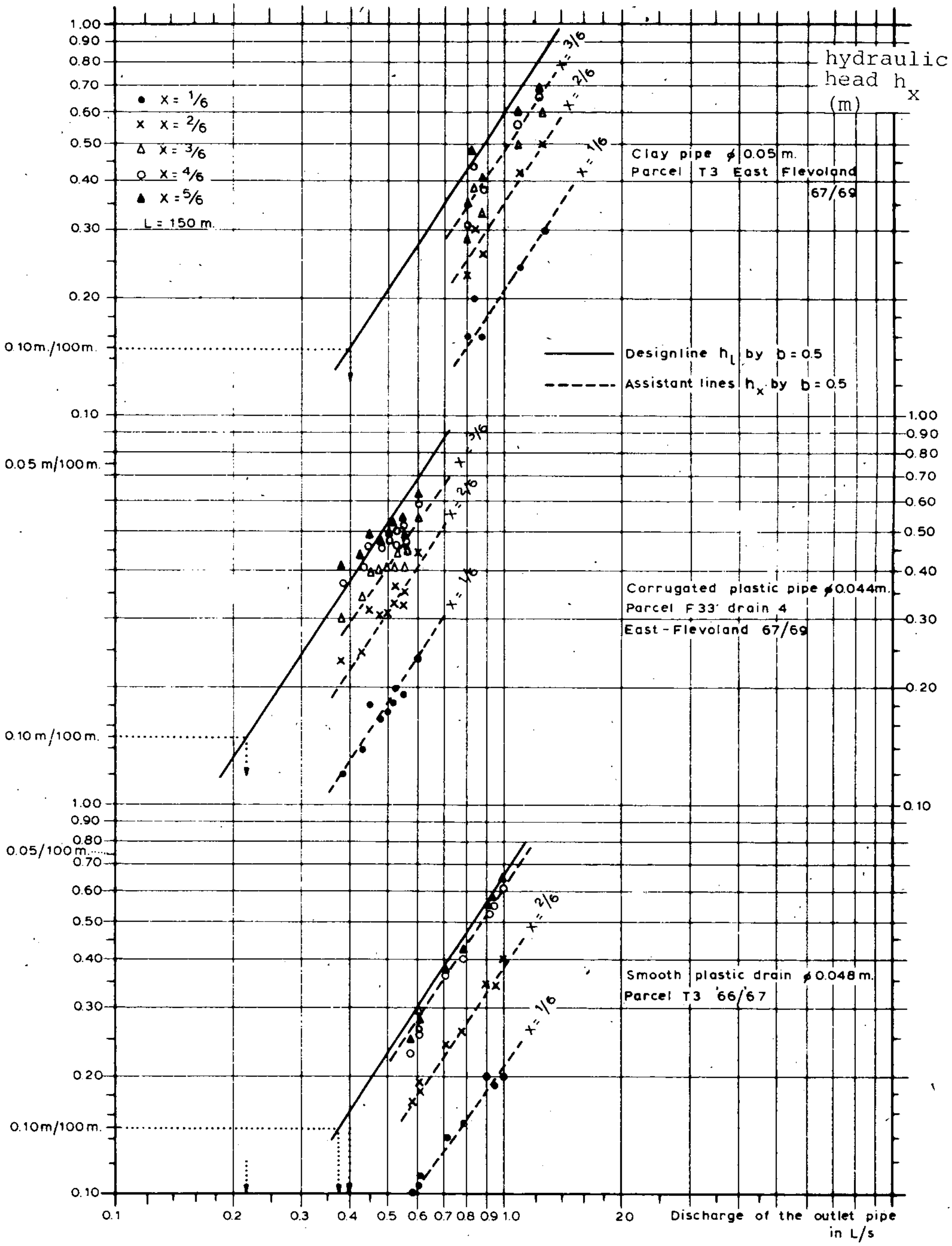
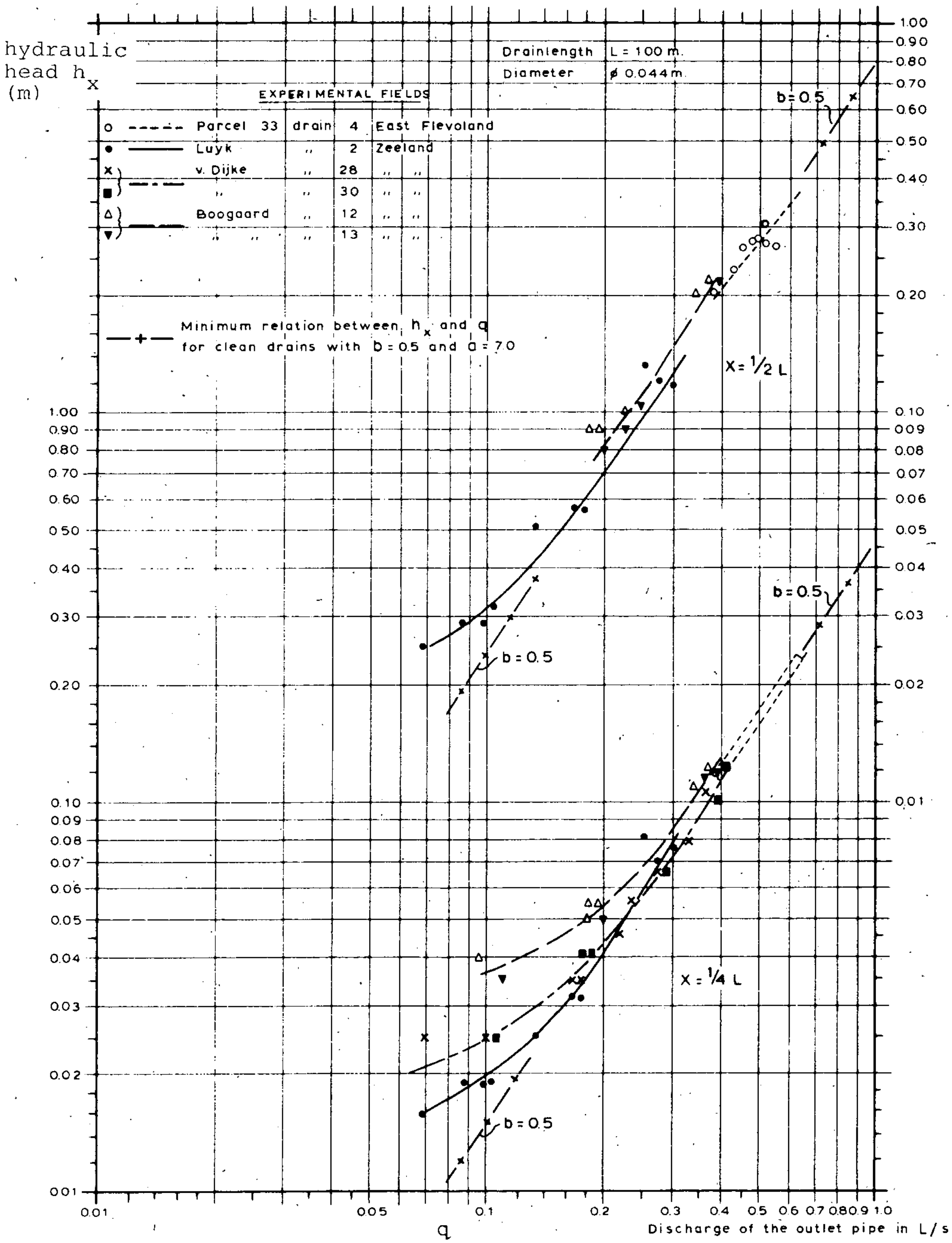


FIG. 2 Relation between hydraulic head and draindischarge of clean corrugated plastic pipes under experimental field conditions in Zeeland and East Flevoland



It should be remarked here that all the data in Figures 1 and 2 pertain to well-maintained clean drains.

For these conditions the field data show that drain discharge capacities can be computed from Eq. 3 with $b = 0.5$ and $a = 4.7$ for smooth plastic pipes, $a = 4.5$ for good quality clay pipes and $a = 7.0$ for corrugated PVC-pipes. Here it is assumed that the mentioned a-values also hold for other pipe diameters.

Laboratory experiments carried out with 58 mm and 115 mm corrugated pipes point into this direction (Hansen, 1975).

3. The effect of maintenance on discharge capacity

From experimental fields in Zeeland with corrugated PVC-pipes (ø 44 mm) it was found that the a-value increased with a factor 1.5 to 2 due to silting up of the drain lines.

This implies that the discharge capacity then has been reduced considerably. In order to be sure of a sufficient transport capacity the drainage system should be designed with a certain over capacity. In other words a certain degree of silting must be allowed without affecting the transport capacity too much. This comes to a reduction factor for the area to be drained by a certain pipe diameter.

The silting generally manifests itself as a segment on the bottom of the pipe. Assuring now a certain silting, the remainder of the cross-sectional free area can be computed. For this cross-section an equivalent diameter can be derived.

Using Eq.4 the reduction in cross-sectional area can be expressed as an increase in the coefficient a. For instance a reduction in cross-sectional area to 90% gives the same effect as an increase in a-value with a factor $(0.9)^{-4.5} = 1.61$.

For different silting percentages the increase in a-value is given in Tab.1. Also the reduction in q, due to the siltage is given in that table.

TABLE 1. Effect of silting percentage on the coefficient a, and the discharge for corrugated pipes (in %)

Silting d/D	Cross-section	Equivalent diameter	Reduction of q with h_L=const.	Increase of a (+h_L) with q = constant
0	100	100	100	100
10	94.8	97.4	92.4	113
20	85.8	92.6	79.4	141
25	80.0	89.4	71.5	166
30	74.8	86.5	64.7	192
40	62.7	79.2	49.7	286
50	50.0	70.7	35.3	476

An increase of a with a factor 1.5 coincides with a silting percentage of 22%, which means that equivalent diameter is reduced to 0.91. When a increases with a factor 2, the silting percentage is 30%. If one is forced to take into account a certain silting danger it is reasonable to apply a larger reduction in drainable area to smaller pipes than to larger pipes.

For the latter the remaining free cross-section at the same silting degree remains. Moreover the danger of silting is smaller because of possible lower flow velocities in the vicinity of the pipe. It is therefore reasonable to apply a smaller reduction to the larger pipe diameters. This may be achieved by allowing a certain constant thickness of the silt layer in all types of pipes. Therefore an increase in a with a factor 2 as found in practice is applied here to 44 mm corrugated pipes. Applying the computations explained for Table 1, this implies that a silt layer of 13.2 mm as an average had to be present.

Introduction of the same layer in other diameter of pipes then lead to a reduction in drainable area as indicated in Table 2.

TABLE 2. Reduction of drainable area as a function of the diameter

diameter	silt layer	silting degree	reduction area
44 mm	d = 13.2 mm	30%	35%
54 mm	d = 13.5 mm	25%	29%
57 mm	d = 13.7 mm	24%	27%
72 mm	d = 13.7 mm	19%	20%
92 mm	d = 13.8 mm	15%	14%

Herewith a reduction dependent on the diameter is introduced which decreases with increasing pipe diameter.

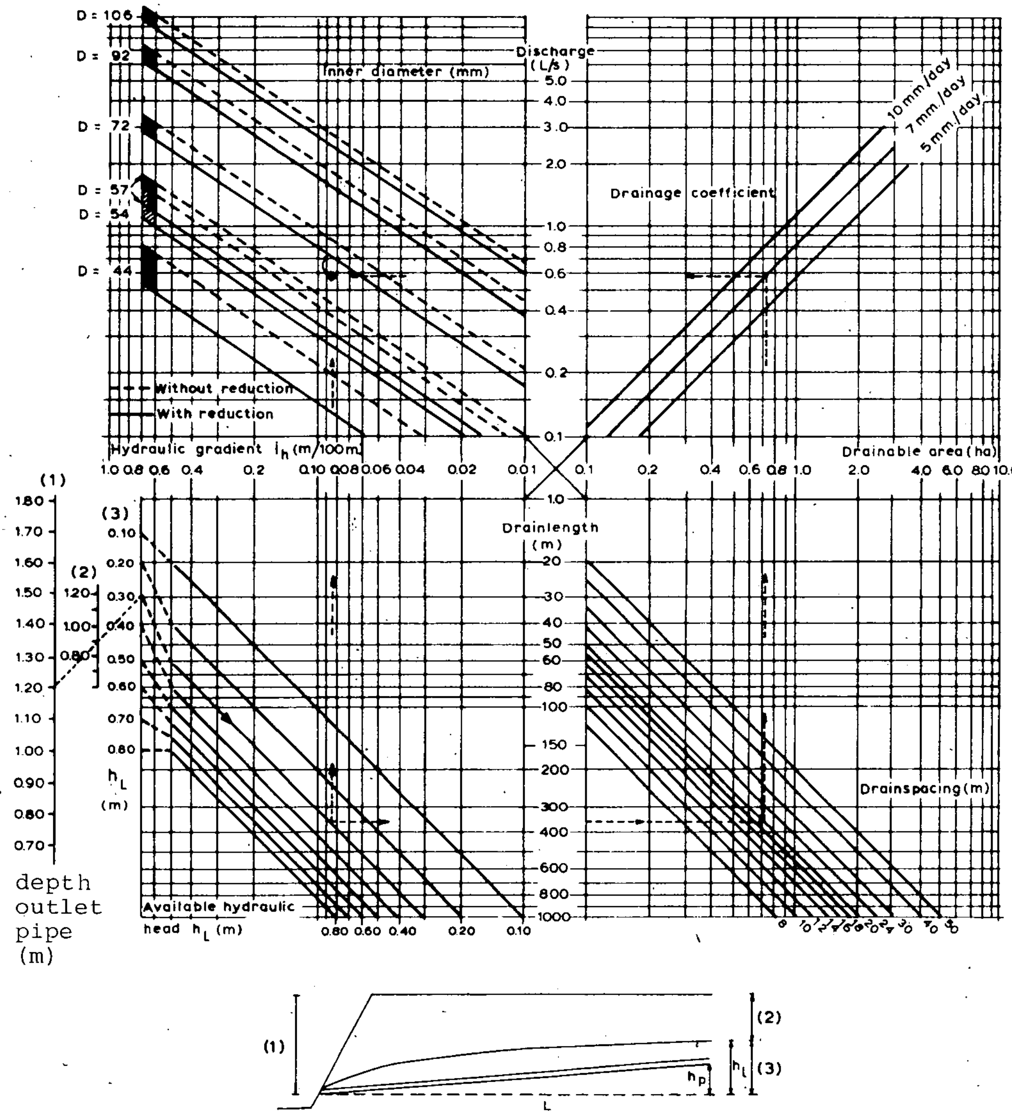
4. A nomograph for the determination of the pipe diameter

Based on Eq. 4 a nomograph for corrugated pipes was constructed in which various design aspects have been brought together (Fig.3).

First of all a distinction has been made between the hydraulic gradient and the slope of the pipes. The hydraulic gradient is the ratio between the hydraulic head difference between upper end and the outlet of the pipe to the total length of the pipe L. The slope of the pipe is the gradient under which the pipe is laid. For design purposes the first property has to be used. If the slope of the drain is not the same as the hydraulic gradient by the design discharge, water will stand above the drain at the upper end of the line. This has certain consequences for the design that will not be discussed here.

The left hand lower part of the nomograph gives the relation between available hydraulic head h_L, drain length L and hydraulic gradient. The lower right hand part is introduced to get the area to be drained. The upper right hand part relates drainable area, drainage coefficient and total discharge. Finally in the upper left hand part the drain-diameters are related to hydraulic gradient and discharge rate. In this part two lines are given for each pipe diameter.

FIG. 3 Nomograph for the determination of the maximal drainable area for corrugated plastic pipes by several diameters and drainage coefficients



Hydraulic gradient $i_h = \frac{h_L}{L}$

Slope of the pipe $i_p = \frac{h_p}{L}$

The first one is based on an a-value of 7.0 valid for clean or well-maintained pipes.

The other line holds in the case a reduction as indicated in Tab.2 is applied, namely for the case a silt layer of about 13 mm is allowed in the pipe.

Lines for other reductions may be introduced parallel to the ones drawn in the nomograph.

Since the hydraulic gradient determines to a large extent the transport capacity of a pipe, longer drain lines may be used when h_L is increased. This means that in general longer lines with the same diameter can be applied if the outlet of the drain is deeper.

In order to have a deeper drainage near the upper end of the drainlines in flat areas it can be advisable to lay the lines as flat as possible

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Paper 2.14

DRAINAGE MAINTENANCE IN THE NETHERLANDS

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Summary

Drain clogging can have a big influence on the efficiency of the drains, both on the discharge and on the hydraulic head (Table 1). For cleaning drains with a length up to 500 meter in general a flushing equipment is used. It is well known that with this equipment sand cannot be removed, but only levelled. The equipment is built up by a hose and a jet. The pump at the other end of the hose causes the water discharge with a certain discharge-head. The amount of water and the discharge-head depend on the used equipment.

The discussions concerned the different types of clogging materials under different circumstances and the frequency of maintenance needed. This frequency varies from once a year up to once every five years. In addition to that, every year the outlet pipes have to be cleaned.

In practice a half to two-third of the drainage systems never have been pierced or flushed, while for about 10% of the drainage systems just rarely this is done.

1. Introduction

To maintain drainpipes is necessary to secure the efficiency of drainage systems. Proper dimensions, sufficiently low ditch levels and such like, ar of course, basic conditions.

In practice, not satisfactory working drainage systems are noticed when water stands in the field for a long time, and, in spring, when the topsoil will remain wet for quite a while.

Sometimes drain discharges are compared with one another. The effectiveness of a drain is seldom discovered by comparing the ground water levels at a point halfway between and beside the drain pipes near the beginning of the pipe.

We shall discuss the maintenance equipment available, how often maintenance has to take place, maintenance costs, how much regular maintenance is actually taking place and the problems to be faced. Finally a number of issues on drainage maintenance will be mentioned.

2. Drain-clogging effects

The dimensions of drainage systems are chosen in such a way that some capacity reduction of the drainpipes because of drain-clogging is taken into account.

Below we shall point out to what extent drain-clogging will reduce their discharge at the same hydraulic head, or how much the hydraulic head must increase to maintain the same rate of discharge.

A study of this matter can be based on the relation:

$$D^{4.75}\ hL = C\ q^{1.75}$$

D is here the inner diameter of the drainpipe (m), hL is the hydraulic head over L metres and q is the outlet pipe discharge (m^3/sec).

The formula $D^{4.5}\ hL = C\ q^{1.5}$ appears to make the relation more clear, but this formula has not yet been published. Besides, it indicates a somewhat worse drain-clogging effect than the first formula.

Drain-clogging is usually caused by a layer of sand or silt settling on the bottom of the drainpipe. If the remaining part of the drainpipe through which the water can pass is thought round, then, starting from its area, we can calculate its diameter and introduce it into the formula. Calculations have produced the following table.

TABLE 1. Drain-clogging effects on the efficiency of a drainline

Extent of drain-clogging				Remaining diameter of drain pipe expressed in x-per cents of the original diameter	Drain-clogging effect	
in mm thickness from the inside bottom of drain pipe with its diameter (expressed in mm)			thickness of layer in relations to drain diameter (stated in %)		a) at the same hydraulic head, discharge reduction down to .. % (rounded off)	b) at the same rate of disch. increase of hydraulic head up to .. % (rounded off)
44	54	57	0	100	100	100
4.4	5.4	5.7	10	97.4	90	115
8.8	10.8	11.4	20	92.6	80	145
13.2	16.2	17.1	30	86.5	65	200
17.6	21.6	22.8	40	79.2	55	305
22	27.0	28.5	50	70.7	40	520

An example will make the effect of drain clogging even more obvious. A PVC-corrugated drainpipe with a diameter of 60 mm, has an inner diameter of 54 mm. If a drain gets choked, for instance, a 16.2 mm layer of sand on the bottom, this will produce a silting effect of 30 per cent. In most designs such a drain will, if clean, give a hydraulic head of 200 mm over 200 m, at a discharge rate of 7 mm per 24 hours. In view of the assumed silting exten a hydraulic head of 400 mm is necessary at a discharge rate of 7 mm per 24 hours.

At a hydraulic head of 200 mm, only a discharge of 65 per cent of 7 mm, i.e 4.5 mm, is possible.

3. Maintenance equipment

3.1 For outlet pipes

Outlet pipes should be clean and be joined to the drainpipes in a straight line. Regular inspection is necessary, so that any failures and stoppages can be corrected by hand labour, with a spade and such like.

3.2 For drain pipes

Labour-intensive cleaning methods in which bamboo rods, thin cables, chains and/of brushes etc. were used, have been replaced by pumping equipment with hoses.

This equipment is used by specialised drainage maintenance concerns, by wage-workers and by individual farmers.

To pierce drainpipes in water disposal periods but without an extra water supply, a torpedo-shaped nozzle is fastened to the hose. To flush drains, the hose is equipped with a jet (see Fig.1). A pump is used to supply water.

The latter method has superseded nearly all other ones. From the inventory of some distributors and 13 drain flushing companies, it has appeared that there is a great variety in the used equipment. This is partly caused by the fact that several parts of the equipment are not very complicated, technically. Figure 2 shows how the equipment is actually used.

a) Pumps

Pumps used to flush drainpipes have very divergent outputs and discharge heads. Figure 3 shows a number of these combinations.

b) Reels

Hoses are wound on reels for easy transport. The reel diameter is usually from 1.80 to 2.00 m. For nylon hoses reels of 0.60 to 1.00 m in diameter are often used.

c) Hoses

The length of the used hoses varies from 200 to 300 m. They are made of very different materials: nylon, tylene, polythylene, polyethylene, rubber, tubylene, tectylene and rubber reinforced with nylon. The first part of the hose must resist the highest pressure and is, therefore, sometimes strengthened with iron of canvas. In some cases a PVC hose is used with a length of 500 m. The permissible pressure in the hose is higher than the pressure produced by the pump.

The inner diameter of the hoses varies from 14 to 20 mm, the outer diameter varies from 18 to 32 mm. The material, the hoses are made of, varies a great deal in thickness. The life of the hoses and the number of times that cracks occur widely differ.

From the available data it has appeared that the type of drainpipe or the type of soil has no influence in this respect. Only a combination of peat soil and PVC pipe, smooth or corrugated, has appeared to grant polythylene hoses a life expectancy of about 10 years. In other cases hoses will last from six months to 4 years, usually 1 or 2 years.

The amount of kilometres drainpipe to be cleaned has appeared to have no influence. Even the number of times that cracks occur seems to be independent of the above-mentioned factors, and differs from scarce to 2 times a week. A PVC hose which has to be replaced twice a year and has to be repaired once in every two days, is an exception when drawing up the inventory, hose material and costs have not been specified. That is why we cannot mention whic hose type is to be preferred. We can state, however, that a PVC hose meets with a low friction resistance of the inside wall of the drain pipe.

The life expectancy of a hose is presumably largely dependent on the care with which it is treated.

When flushing drains the pressure in the hose will reduce. Apart from loss of pressure caused by bends and such like, this reduction in pressure will be for a hose with an inner diameter of 20 mm and a length of 200 m about 30 atm, at a pumping capacity of 50 l/min., about 120 atm. at a capacity of 100 l/min. and about 260 atm. at a pumping capacity of 150 l/min.

To be calculated by means of: $0.1\ \lambda \frac{L}{D} \frac{V^2}{2g}$ (atm.) in which: $\lambda = 0.08$ (max.), $L = 200$ (m), $D = 0.02$ (m), $2g = 19.6$ and $V^2 = Q^2 : A^2$.

d) Nozzles

The nozzle diameter varies from 25 to 45 mm, but most nozzles have a diameter of 28 - 30 mm (see Fig.1).

One of the jets from the nozzle is usually directed forward and 3 to 6 other ones are directed obliquely backwards. Sometimes there are even more jets, this seems to be preferred for sandy soil.

3.3 Application

The degree of atmospheric pressure which is used when flushing drain pipes, JET

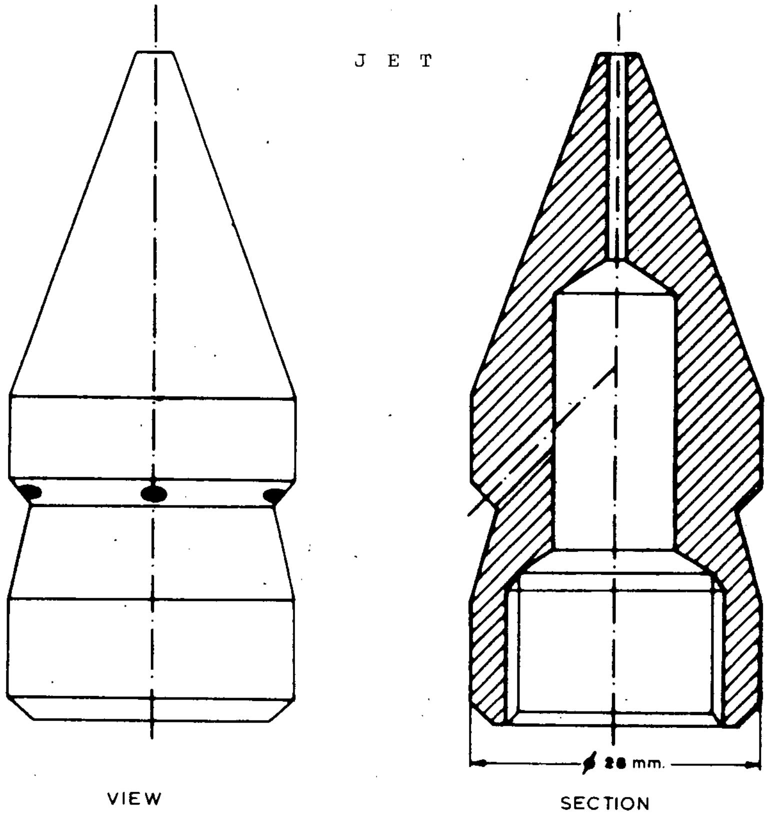
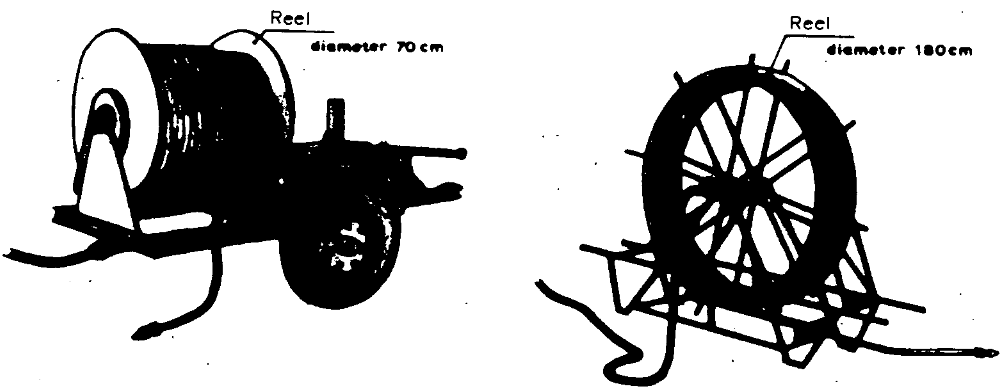


Fig. 1



FLUSHING EQUIPMENT

Fig. 2

should also be dependent of the type of soil round the drainpipe. In light sandy clay- and sandy soils the pumppressure should not exceed 30 atm, in connection with a possible silting of the drainpipe during and perhaps even after flushing operations.

From inventories it has appeared that higher pressures are certainly applied in such soils. To this, we can add that the pressure in the pump is much higher than the pressure in the nozzle.

The average length of drainlines to be flushed is 150 m, in some areas it may run up to 200 m. The average drain pipe diameter is 50 to 50 mm.

With about 50 per cent of the companies drain-flushing is a one-man job. The other 50 per cent employ two men.

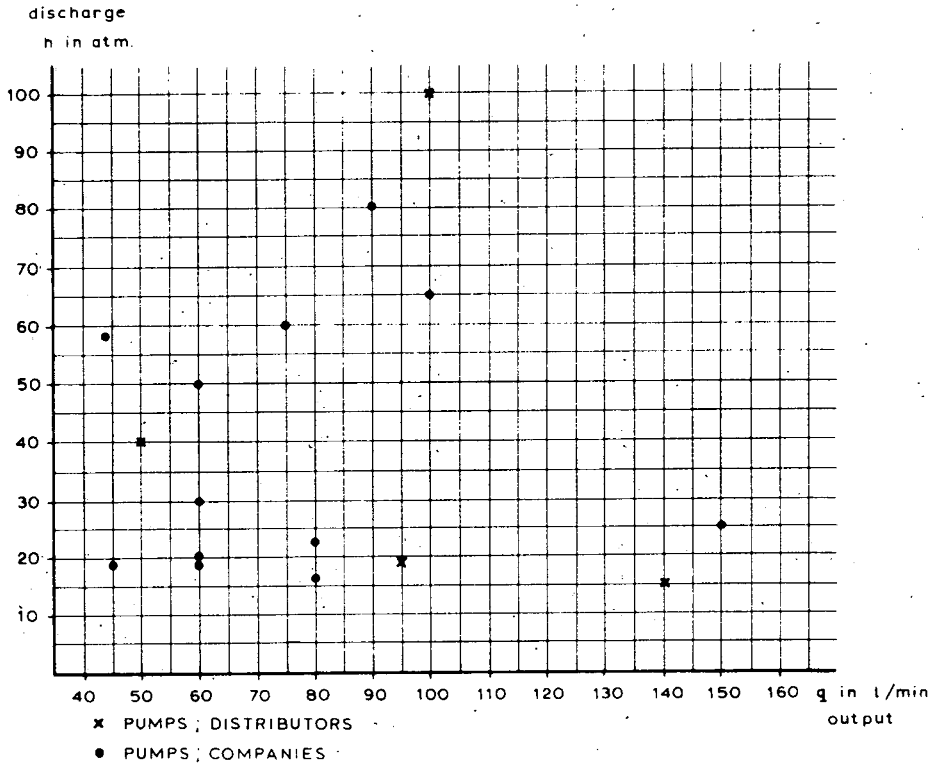


Fig.3. Pump types.

4. Drain pipe clogging

4.1 Some general notes

The most common causes of damage done to drainage systems are: iron compounds, sand, mud and root growth.

In all types of soil one or more of these can occur, dependent, among other things, on hydrologic circumstances and the way the soil is used.

Beside removing all that which causes damage to the system, the purpose of drainage maintenance is also to locate and repair defects such as: broken, dented or sagged pipes, loose joints and, in case of outlet pipes, vegetation growth and disturbances. These drain defects can occur in all types of soil.

Here follows a survey of drain failures occurring in different types of soil.

4.2 Light sandy clay and sandy soils

The drainlines in these soils generally require most maintenance. Most drain failures in these soils are caused by iron compounds and sand being washed into the drains. Iron deposits are hard to prevent. To keep the drain pipes permanently submerged will help to avoid such a deposit.

Sand washed into the drains is mainly caused by draining under adverse circumstances (a high groundwater level) and/of the use of inferior filter material which is not fit for the soil in question. Other causes of drain failures which can occur in light soils are:

sulphur compounds (in salty or brackish soils) and root growth, particularly tree roots in woodland, strips of land planted with trees and shrubs and windbreaks.

The drain trench environment in light soil is often considerably more favourable for root growth because of a better supply of oxygen than intact soil.

Silt, clay, humus or a such like deposit in the drain trench highly stimulates root growth.

4.3 Heavy sandy clay- and clay soil

These types of soil are generally the least complicated as for drainage maintenance. Iron deposits rarely occur and root growth is found much less often than in light soil. In heavy soil containing salt or brackish groundwater, lutum transport can take place resulting a deposit of silt in the drain pipes.

4.4 Peat soil

In peat soil a deposit of iron compounds can be found. The chance is fairly large that roots will grow in drain pipes laid in woodland.

4.5 Areas raised with sand (for house-building, etc.)

In these areas there is a great chance of iron deposits, especially during the first few years after a drainage system has been installed.

4.6 All types of soils

Drain pipe failures which can occur in all types of soils or under certain hydrologic circumstances, are caused by:

a) Outlet drain pipes getting fouled or choked up with overhanging grass or weed and root growth. These plants are, in turn, easy targets for iron compounds to get attached to.

b) Iron deposits in the drain pipes brought down along with seepage water. This deposit depends on the flow rate of the seepage and the amount of iron the seepage water contains.

5. Maintenance

5.1 Drain pipe failures

About the various kinds of drain pipe failures the following general statements can be made:

a) Iron- and sulphur compounds

In so far as these compounds are found in drain pipes they can be easily removed by piercing the pipes when there is a sufficient water discharge or by flushing, provided that cleaning activities are not put off for too long.

Seepage water can easily choke up the pipes again. Iron- and sulphur deposits in filter material or between filter material and drain pipes are very hard to remove with the existing methods.

b) Sand washed into the drains

It is very difficult to remove sand from drains. Piercing and flushing drains (even at a high pressure) will only result in moving the sand a little way.

If the sediment is such that the nozzle can hardly get through, the risk is great that more sand will be deposited in the drains. A labour intensive but, according to experts, effective method to remove sand from drains, in former use, was to draw a chain or scraping springs through the pipes. Small gauges are also very useful.

c) Drain pipe silting

Silt is more difficult to remove from drains than iron compounds. If it is still fresh, it can be washed out by flushing at a high pressure if necessary.

If the silt has dried up, then it takes much trouble to clean the pipes. Therefore it is to be preferred to flush the drains when they are discharging water.

d) Roots

Crop- and weed roots can easily be removed by piercing or flushing the drains, the more so as most of these roots die off in wintertime. Tree roots cause more problems, particularly when they are not removed in time.

Root growth can be prevented by laying the drain pipe in woodland so deep, that in summer at least, they are permanently under water. Under windbreaks non-perforated pipes can be used.

5.2 The inventories of maintenance concerns

From the above mentioned inventory of a number of drain-flushing companies in various parts of The Netherlands, the following appears:

a) Drain pipe cleaning

Even if it appears that the entire length of drain pipes can be flushed there will still be sand left in the drains, as most people know. A few times it has been reported that iron had been left behind as well. It has appeared that not all drain lines can be entirely flushed. The number of these drains varies from 1 to 5% of the total number, in the various areas. This is caused by sand being washed into the drains and sometimes by root growth.

b) Drain pipe defects

The fact that the pipes cannot be entirely pierced, is also caused by defects in the drain pipes. Their frequency appears to differ widely. One to nearly 15% of the number of drain lines appears to have a defect its average being approximately 5%. From data material available tile drains or PVC pipes are used. They will be repaired when they are flushed or the defect will be reported to the land user.

c) Outlet pipes

Many outlet pipes appear to need repairs. Their number varies from one to three quarters of the total number, depending again on the area in which they have been laid. In particular concrete and eternite outlet pipes appear to raise difficulties. Also the various types of plastic outlets may sometimes cause problems.

5.3 Maintenance results according to field work observations

A few general experiences will be mentioned here:

- <16 mu particles can be removed easily by means of drain flushing, the amount is not too serious;
- rust can be removed. When there is a very strong inflow, the pipes need frequent flushing;
- sand cannot be removed by flushing drains, or only over a distance of 10 or 20 m from the outlet pipe;
- great quantities of water which cannot flow back through the outlet pipe, because the drain pipe between the outlet pipe and the nozzle has got choked up during flushing activities, may find their way to the surface;
- the extent in which extra silting still occurs in light sandy clay soil, after drain flushing at a high or low pressure, is assumed to be partly dependent on the stability of the soil. When considering the matter the following should be kept in mind: drain pipe environment - at a suspensible fraction percentage (< 16 mu) or over 25% there is no silting danger; at lower percentages extra silting is possible.

6. Maintenance frequency as it is desirable under different circumstances

6.1 In sandy- and light sandy clay soil

farming areas, without seepages,
piercing or flushing at a low pressure,
1st year after drainage has been installed,
3rd or 4th year, 7th or 8th year, then every 5th year

farming areas, with seepages,
piercing or flushing at a low pressure,
1st year after drainage has been installed,
then every 2nd or 3rd year dependent on the flow rate of the seepage and the amount of iron the seepage water contains.
If there is a seepage, it is to be recommended to pierce the first 10 m of drain pipes from the outlet at least once a year

woodlands, with or without a seepage,
piercing or flushing at a low pressure,
once a year, if possible

6.2 In clay- and heavy sandy clay soil

farming areas, without seepages,
piercing or flushing at a low pressure,
first year after drainage installation, then every 5th to 10th year

farming areas, with seepage,
piercing or flushing at a low pressure,
first year after drainage installation,
then every 2 or 3 years (see: sand- and light sandy clay soil)

woodlands,
piercing or flushing at a low pressure,
first year after drainage installation, then every 3 or 4 years

6.3 In peat soil

farming areas without seepages,
piercing or flushing at a low pressure,
first year after drainage has been installed, then every 3 to 5 years

farming areas, with seepages,
piercing or flushing at a low pressure,
first year after drainage has been installed,
for more information see: sandy- and light sandy clay soil

woodlands,
see: sandy- and light sandy clay soil

6.4 In raised areas

flushing at a low pressure,
first year after drainage installation, 3rd year, then every or 4 years or more frequent, if much silt is removed by flushing

6.5 All types of soil

Inspection of outlet pipes. Removal of overhanging vegetation.
Drain piercing over a length of 10 m. Every year before mid-October.

6.6 Actual maintenance frequency

The companies whose inventories have been drawn up, suspect that half to two-thirds of the drainage systems have never been pierced or flushed and 10% rarely.

They carry out maintenance work, once every 10 years, once every 5 years, and once every 2 years, respectively 0-20, 0-20 and 0-10% of the number of drain lines.

7. Maintenance costs capacity

The costs of flushing drain pipes vary from D.fl. 0.08 to D.fl. 0.50 per metre, the average being D.fl. 0.15.

If the costs are calculated per hour, the price will be D.fl. 40,-- to D.fl. 45,-- per hour. From this it follows that about 300 m can be flushed every hour.

This is, of course, largely dependent on the degree of silting. Drain pipe repairs are always calculated per hour. The extent of these works cannot be estimated in advance.

8. A few points at issue

Drain pipes

If drain pipes are laid with sufficient care, they can be laid tight, so that they are easy to maintain. Saggings at a later stage may induce repair.

Outlet pipes

Outlet pipes appear to require much care. A satisfactory type of outlet pipe does not exist.

Outlet pipes with gutters kept inside the side slopes are probably the best alternative at the moment.

Changes in the soil structure owing to flushing operations. It is sufficiently known whether, and if so, to what extent, the structure of the soil around the drain pipe changes - deteriorates - because of flushing operations.

Especially at high pressure pumps, great quantities of water and a stagnant nozzle, the soil near the drain will be soaked through. The effect this may have on silt being washed into the drains and inflow resistances are largely unknown.

Perforations - butts - wrapping material - root growth

As we mentioned before, materials being washed into perforations, but joints and wrapping material, is very hard to remove, if not impossible.

Especially when thick-walled pipes and bulky wrapping material has been used. Tree roots can only, if at all, be removed at an early stage.

Drain pipe silting

Grains of earth < about 0.05 mm (= 50 mu) washed into the drains can be removed by flushing the drains.

Also iron is easy to wash out. Sometimes the iron deposit grows so fast, that the drains must be flushed again after a few weeks.

Drain sanding is one of the greatest and most frequent problemts of drainage maintenance.

It is generally assumed that sand cannot be flushed out of the drains. It will be moved a little way.

If, in that case, no shoals form in consequence of a stagnant nozzle, the layer of sand will only be levelled.

Experiments with different diameters of hoses have shown that in the best, sand only can be removed for a distance of 20 m from the outlet pipe.

Paper 2.15

GUIDELINES FOR THE SELECTION OF ENVELOPE MATERIALS FOR SUBSURFACE DRAINS

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Summary

Whether an envelope material is essential or whether the pipe can be left unwrapped depends on the type of soil and the laying conditions. From the results of the laboratory and field research in recent years by the Government Service for Land and Water Use, the Institute for Land and Water Management Research, and the IJsselmeerpolders Developments Authority, the following conclusions and recommendations have been formulated. (See also Tables 2 and 3.)

1. In non-cohesive soils that are unstructured and unstable, one must be prepared for continuous sand inflow and pipe sedimentation. This requires a durable material (voluminous or thin). Thin materials are effective if the soil has a low clay particle and humus content and if no chemical and bacterological deposits are expected.

2. In sandy loam, loam, and silt loam, temporary instability must be expected. In these soils good results are achieved by covering or wrapping the pipe in organic material. In sandy loam and silt loam, special care must be taken to ensure that the trench is properly filled; otherwise excessive demands are made on the selective filtration function of the (voluminous) material. The materials at present available are on the whole unable to meet such demands.

3. In light and heavy clay soils there is no need to cover or wrap the pipes

4. At present the materials available for land drainage are mainly limited to organic materials: coconut fibre, a mixture of peat (75%) and coconut (25%), peat fibre, and flax fibre used to a lesser degree.

On the basis of laboratory and field research, polystyrene granules in perforated plastic sheeting and polypropylene fibres (fitted round the pipes like coconut fibre) can be recommended as suitable voluminous synthetic materials, provided they are at least 7 mm thick. Research into the use of thin synthetic materials has not yet been completed. The KOMO Filter Material Committee has established quality standards for the organic materials peat, coconut, peat/coconut fibres (KOMO, 1976) and is preparing similar requirements for synthetic materials.

Introduction

This article is based on the results of laboratory research conducted by the IJsselmeerpolders Development Authority (RIJP) and the Institute for Land and Water Management Research (ICW), and on field investigations by the Government Service for Land and Water Use (LD) and the RIJP.

Developments in materials

Ever since tile drains have been used, they have, wherever necessary, been covered or surrounded by organic or anorganic materials. "Wherever necessary" means in those soils where the soil particles tend to migrate towards and into the pipes. The materials used were straw, straw and dried heather, peat fibre, gravel, and other natural granulates which were loosely applied to the pipes. The introduction of corrugated plastic tubing in the mid-sixties, combined with the steep rise in labour costs and the advances being made in the mechanization and automation of drainage work, made it necessary to develop cheaper and more labour-saving envelope materials that could enhance further mechanization and speed up the pipe laying techniques. In the mid-seventies the stage was reached that all pipes supplied to projects requiring envelope materials arrived on the site pre-wrapped by the factory. The intermediate stage of envelope materials being supplied to the site in strip form belongs definitely to the past.

The materials used for the manufacture of the new pre-wrapped envelopes are the organic materials also used formerly (straw, peat litter, and coconut fibre) and also synthetic products. The synthetics preferred nowadays are of a voluminous type: polypropylene fibre mats, acrylic fibre mats, polystyrene flaxes contained in a polyvinylalcohol fibre membrane, polypropylene fibres (the synthetic counterpart of coconut fibres), polystyrene granules held around the tubing by a perforated plastic sheeting, with thicknesses varying from 3 to 10 mm.

Apart from these comparatively new products, glass fibre, woven and non-woven nylon membranes, and other fleeces are still manufactured and occasionally employed.

Functions of an envelope

When making a choice of envelope materials, one should remember that the envelope is required not only to prevent certain soil particles from entering the pipe (its selective filtration function), but also to improve the water flow into the pipe (its hydrological function). We refer to the first as the selective filtration function to make it clear that the envelope must always let through the soil particles suspended in the water; otherwise the material would become clogged in the course of time. Whereas finely structured materials are preferable for filtering, voluminous coarser-structured materials are preferable for the hydrological function. The soil, however, is composed of particles of every possible size, and therefore always contains particles that will block up the mesh of finely structured materials. Particles may also coagulate in the proximity of the drain as a result of the precipitation of iron compounds, etc., or these complex iron compounds may themselves block up the mesh. In practice, therefore, voluminous materials with a fairly coarse structure are found to be the best.

A recent study by Nieuwenhuis (1976) showed that where the outside of envelope materials had become partially blocked, this had very little adverse effect on the approach flow resistance, provided the material was about 7 to 10 mm thick. It has generally been found, however, that this depends greatly on the type of soil that is being drained.

Thin membrane-type materials can do an excellent job in moderately fine to coarse sandy soil. As the fine-to-very-fine particles are flushed out and the coarse particles are retained by the membrane, a natural soil filter is built up in the vicinity of the pipe and its wrapping, thereby preventing any more fine particles from entering the pipe. This has been demonstrated in the studies of Ogink (1975) and Chrissanov (personal communication, 1976) and others. It is therefore obvious that the required structure and thickness of envelope materials depend very much on the type of soil.

Factors that determine the use and choice of envelope material

Three factors determine whether an envelope is required and, if so, which material should be used. These factors are:

a) soil factors
b) hydrological aspects
c) laying conditions.

a) Soil factors

The risk of soil particles entering the pipes depends on:

- the soil texture (granular composition)
- the soil structure and stability (cohesive forces).

Soil particles of 0.05-0.15 mm that enter the pipe and are deposited in it are very difficult to remove. Particles smaller than 0.05 mm can hardly be kept out by envelope material. Any envelope fine enough to do so would eventually become clogged. Besides, such particles usually remain in suspension and are carried away by the flow of water. Particles larger than 0.15 mm present few problems, because the velocities and hydraulic gradients of the flow in the vicinity of the pipe are insufficient to transport them.

Soil stability has been expressed in the past by uniformity coefficients. To describe the aggregate stability, sherard (1953) supplemented this with the plasticity index, as shown in Table 1.

TABLE 1. Soil stability expressed in uniformity coefficients and plasticity index

Uniformity coefficient D60/D10 1)	Plasticity index PI 2)	Soil stability
< 5	< 6	low
5 - 15	6 - 12	medium to high
> 15	> 12	high

1) D10: mesh width letting through 10% of soil particles by weight.

2) PI : the plasticity index is the difference in moisture content between the upper and lower limit of plasticity of the soil (Hofstee and Fien, 1971).

To determine the stability of cohesive soils, Olbertz (1965) took as his basis the findings of Dunn (1959) on the ratio between the force required to transport soil particles and the size of the particles, and supplemented them with the ratio between the percentage of clay particles (smaller than 0.002 mm) and the percentage of silt and clay (particles smaller than 0.02 mm). If this ratio is less than 0.5, the cohesive soil should be considered unstable.

b) Hydrological aspects

Drainage designs are based on the presupposition that there is no resistance to the flow of groundwater towards the perforations in the drain because the soil in the drain trench is assumed to be more permeable than the undisturbed soil. In low stability soils this assumption is not correct if the pipes have been laid under wet conditions. In such cases, however, it is possible to reduce the approach flow resistance placing a highly permeable medium round the drain.

In infiltration areas it is equally essential to keep exit resistance to a minimum; hence only voluminous materials are used in those areas. Experience in infiltration areas in the North East Polder has revealed that peat fibre in an environment which is alternately wet and dry (reduced conditions in summer and more or less aerated in autumn, winter, and spring) is subject to accelerated decomposition. Consequently this material is no longer used. Instead, dried heather covered with a thin layer of straw is employed.

c) Laying conditions

Unfavourable conditions for pipe laying are those in which the soil is wet as a result of rain or high groundwater and is therefore to some extent (depending on the type of soil) unstable. This impairs the efficient functioning of the drains. Van Someren (1965) pointed this out as early as 1965, and it has been fully confirmed by field research since then (Cavelaars 1966, Segeren and Zuidema 1969). Unsatisfactory drain functioning can frequently be traced back to wet laying conditions. Subsoils of loamy fine sand, fine sandy loam, and fine sandy clay are particularly sensitive; with these soils there is a likelihood of particles entering the envelope material and the pipe, and a danger of the soil around the pipe solidifying, or at least becoming less permeable. In sand, silt, and loamy sand, the greatest danger is that of sedimentation, and the main function of the envelope material is that of selective filtering. Sandy loam and silt clay loam are particularly prone to structural deterioration, and if pipes are laid under wet conditions the trench backfill shows a severe reduction in permeability. These soils, however, unlike the fine sandy soils mentioned above, are more likely to recover their permeability during subsequent dry periods. In loamy sand, sandy loam, and silty clay loam there is also a great sedimentation hazard; if voids are left in the backfilled trench or by the trenchless machine rainfall or surface runoff may find its way to the drain, through these voids, transporting with it large quantities of soil particles towards and into the drain. The envelope material must be able to cope with this without becoming clogged. It is better, however, to prevent the problem by filling the trench carefully under good conditions and tamping the top 30 cm.

Covering drains with top soil

Generally speaking, the practice of covering drains with top soil is incorrect. Experience has shown that after the top soil has been placed on the drains, it sometimes changes into a consolidated, somewhat acidic and impermeable layer. In a recent investigation into the decomposition of coconut fibre (Meijer and Knops, 1977) it was also discovered that when top soil containing humus is brought into direct contact with the fibre, its decay is accelerated. Where the subsoil is immature, the trench should be filled with well-matured soil, but care should be taken not to bring any top soil into contact with the drain.

Choosing the right type of material

In deciding which type of material to use, a choice must first be made between the following alternatives:

a) cover or envelope material
b) fibrous or granular
c) voluminous or thin
d) organic or synthetic.

a) Cover or envelope material

This question hardly arises nowadays, because it usually costs no more to wrap a drain than to cover it, and wrapping is preferable in very many instances where the material has to perform both the hydrological and the selective filtration function.

b) Fibrous or granular

Most of the materials in use in The Netherlands are fibrous. Natural granular materials such as sand and gravel mixtures, shells, blast furnace products, etc., can hardly be considered for land drainage because of the high costs of transport and laying, and the problem of transporting such heavy substances across the land. They are only economically viable for the drainage of urban areas and sports fields.

c) and d) Voluminous or thin, organic or synthetic

These two choices cannot be considered separately, because an organic material is usually voluminous, while some of the synthetic materials are thin and others are voluminous. A thickness of 7 mm or more is considered voluminous. Voluminous materials include peat fibre with gauze, and fibres of coconut and flax, either in strips or as prewrapping. Of the current synthetic materials, polystyrene granules in plastic sheeting and polypropylene fibres also belong to this group. So of course do granular materials that are deposited in loose form, such as peat fibre and dried heather used in combination with straw. Thin materials, most of which are synthetic, vary in thickness from several tenths of millimetres (glass fibre) to several millimetres (Cintamat). Several voluminous and thin materials which were found to be the most promising in laboratory tests are currently being tried out in the experimental fields of the LD and the RIJP to determine their practical value.

After analysing the data from numerous experimental fields of the LD, Jonkers and Miedema (1975) concluded that voluminous materials of comparatively coarse structure are the most suitable for reducing flow resistance. Laboratory and field research by the RIJP (Scholten (1974)), however, have shown, that in very fine sandy soils such materials afford an inadequate guarantee against pipe sedimentation. Nonetheless, provided the requirements as to homogeneity and minimum thickness are strictly observed, voluminous materials are considered the most suitable for these soils too. By and large, although it can be concluded from the findings of these researchers that voluminous materials are the most suitable, thin materials can do a good job in moderately fine to coarse sandy soils (sand and clayey sand) and loamy sand, provided there are no problems of deposits (iron, sulphate, etc.) caused by chemical and bacteriological processes.

Quality standards for cover and envelope materials

With non-synthetic materials, experience has shown that the quality of the end product depends on the origin and processing of the natural fibres, and that consignments vary greatly in quality according to the purchaser's requirements and the price he is prepared to pay. To ensure that the vast sums invested in drainage produce the desired effects, poor quality materials should never be used. To guarantee that they are not, government bodies have established quality standards which have been applied on subsidized land drainage projects since 1974. These standards cover peat, flax, and coconut fibre in both strip and envelope form. Under the auspices of the KOMO[1], the Filter Material Committee recently completed its work on quality standards for peat, peat/coconut (75%/25%), and coconut fibre wrappings.

To arrive at comparable standards for synthetic envelope materials, research is at present being conducted into their density requirements dependent on soil type.

Which material to use in which soil

Cohesive soils with a clay particle content of about 25% or more, which include loam soils, light and heavy clays, and clayey loams, do not usually present instability problems and are not particularly sensitive to structural deterioration of the trench backfill as a result of wet laying conditions.

Soils with a clay particle content of 20 to 25% may be subject to temporary instability as a result of wet laying conditions, but since they quickly recover their stability, it is usually sufficient to cover the pipes with a good organic material.

Van Someren, Scholten, and Zuidema based the recommendations and suitability assessments given in Tables 2 and 3 on the laboratory and field research by the ICW, the LD and the RIJP. Table 2 was compiled on the assumption that it is better to use prewrapped pipes than to cover pipes with strips because covering does not prevent sediment-inflow from below (Busser 1977).

At present there is only a limited choice of materials. The organic materials are: peat, peat/coconut (75%/25%), coconut and flax fibres. The voluminous synthetic materials are: polystyrene granules in perforated plastic sheeting and propylene fibres, both with a minimum thickness of 7 mm. Cintamat, which

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[1] *The Foundation KOMO is the official Dutch organization for the control of quality standards of materials and constructions.*

TABLE 2. Recommendations on what cover or envelope material to use

Soil type at drain level (Textural classification by Netherlands Soil Survey Institute, roughly translated according to USDA classification)		Cover or envelope material: envelope (E) cover (C)	Cover or envelope material: voluminous (Vol) thin (Th)	Comments
clay/loam	medium to heavy heavy clay (mature)	none	-	Where the subsoil is immature, fill the trench with well-matured soil, preferably not top soil.
clay/loam	unstable, immature clay or sandy clay (e.g. resulting from seepage)	E or C	Vol	-
clay/loam	light subsoil, unstable in places	E	Vol	-
sandy loam (possibly fine)	sandy loam or sand	E	Vol or Th	-
brook or rock loam	fine sandy soil containing silt or loam	E	Vol	-
clayey sand and sandy loam containing humus	clayey sand and sandy loam	E	Vol	If drains are also used for subsurface infiltration, do not use peat fibre or thin materials (1).
sand, but not very fine	sand, but not very fine	E	Vol or Th	If drains are also used for subsurface infiltration, do not use peat fibre or thin materials (1).
building sites raised with sandfill	unstable sand above or below drain	E	Vol	"Lavalite", gravel/coarse sand are also possible, provided they are well graded.
reclaimed peat	mixture of sand and peat	E	Vol	Use drain type with large perforations (size class: 1.4-2.0 mm).
peat	degenerate, amorphous or pulverised or clayey peat	E or C	Vol	Use drain type with large perforations. Avoid laying drains in severely decayed, immature peat.
peat	non-degenerate, more or less coarse structure	none		Use drain with large perforations. Good results without E or C.

(1) In these soils it is vital that no top soil containing humus be put on the drains

TABLE 3. Assessment of the suitability of cover and envelope materials

Qualities required	Voluminous material	Thin material
Suitability as regards low resistance to flow	Peat, peat/coconut, coconut and flax fibres, dried heather covered with a thin layer of straw are all good. Oat or rye straw are fairly good. Polystyrene granules and polypropylene fibres are also good.	Depends on density of material and drainage conditions. Acryl and polypropylene fibre mats, polystyrene granules have so far been found moderately satisfactory. Woven nylon and glass fibre poor. Present greatest dangers in fine sandy soil containing humus. Best suited to low-humus medium to coarse sands with low clay particle content and no iron compounds. Do not use in soil where groundwater contains iron.
Suitability as regards preventing drain line sedimentation	When used in light sandy soils, these materials are not, generally speaking, altogether effective in preventing drainline sedimentation. In order of diminishing effectiveness they are: peat/coconut fibre, peat, coconut fibre, polypropylene fibres, polystyrene granules in sheeting, heather covered with a thin layer of straw, flax and straw. It is now believed that materials of greater density are preferable for fine sandy soils, despite their slightly higher resistance to flow. Well-graded sand and gravel or blast furnace products are very effective, but only economically viable for special-purpose drainage.	All good.
Durability	The organic materials are not durable; of the above, peat is the most durable. Decomposition speed depends very much on the circumstances. In soils where only temporary filtering is required (sandy loam and silt clay loam), limited durability is of no consequence.	Polypropylene, polystyrene and polyethylene fibres have unlimited durability in the soil. Acryl fibre is slightly sensitive to high pH. In the long term nylon and polyvinylalcohol would dissolve in the soil water.

in its present form is one of the thin materials, may in the future also mee this requirement. It remains to be seen if these synthetic materials will be available at competitive prices. Current research will determine the potenti use of thin materials like acrylic fibre mats and various makes of polypropylene fibre mats.

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Paper 2.16

BACTERIAL ACTIVITY AND IRON COMPLEXATION AS DRAIN CLOGGING AGENTS

(To be published in: Transact.of the.Am.Soc.of Agric.Engng under the title: Fundamental characteristics of slime clogging in drainage and irrig.systems.)

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Abstract

The most familiar drain clogging problem has been the highly visible Fe sludge called ochre which is caused primarily by the bacterial precipitacion of Fe^2. Studies with chemically precipitated iron have usually been conducted in laboratories using waters devoid of the complexing agents present in drain outflow. Iron complexing with such agents as tannins and humic acids often remain in solution in canals and ponds and there seems to be little information on the role of complexing agents in relation to ochre formation. Iron precipitating bacteria of the genera *Leptothrix, Gallionella, Sphaerotilus, Naumaniella, Toxothrix, Pseudomonas* and *Enterobacter* have been grown in monitored bacterial growth chambers resembling drain lines. Ochre developed only when the bacteria were present. Tannic acid, citric acid, lactic acid, and sulfonated ligning stimulated ochre formation when injected at 1 ppm and such compounds are known to be present in groundwater in trace amounts.
The clogging characteristics of chemically oxidized iron was separated from biologically precipitated iron by injecting biocides and Fe^2 into bacterial growth chambers. A continuous injection of 0.5 ppm free residual chlorine oxidized Fe^2 in the flowing water and killed bacteria with no subsequent ochre formation. The Fe^3 precipitated by NaOCl did not stick to glass slides, beakers, polyethylene or PVC tubing.
Complexing agents may contribute to clogging. Suspended colloidal organic matter can stick to glass slides and bacterial growth chambers and will appear red when stained with carbolfuchsin. Organic matter sorbs and apparently oxidizes iron so that the resultant deposit is a thin, translucent tan. The deposit is usually thinner than gelatinous ochre. Certain complexation products such as citric acid and iron will stick to glass slides, polyethylene and PVC in bacterial growth chambers. Additional studies are needed to establish the extent to which humic acid, tannins, lignins and other naturally occurring complexing agents with Fe can form particles capable of sticking in the zone abutting the drain envelope.
Additional methods are needed for estimating ochre potential before installing drain lines. While Fe^2 flowing into drains is essential for ochre formation, there are other factors such as pH, soil type and temperature that should be considered. There is still no reliable way for predicting the severity of ochre formation since the biotic factors at any given location can only be assumed and not evaluated in advance of drainage operations.

Paper 2.17

IRON CLOGGING: DIAGNOSIS AND THERAPY

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Summary

Knowing the causes of chemical and biological iron clogging and the sites where this may be expected are important for counter-measures. Both can be determined by an orohydrographic analysis of the site and field or laboratory tests on Fe^{++} content and pH in water and soil (Tables 3 and 4). Withi a 20 years research program it was found that: Allochthonic clogging (alien origin of iron) is a continuous problem. Counter-measures therefore show only short-term effects. Continuous maintenance (mechanical and chemical cleaning) remains necessary; Autochthonic clogging (indigenous origin of iron) is only a temporary problem. It depends on soil ripening with time of drainage (3 to 4 years). The iron oxidation within the soil can be enhanced by subsoiling and liming. If iron clogging problems are expected, filters should not be used, except special Antoc-filters, and with submerged drain outlets. Plastic pipes develop adhesion forces for iron hydroxides. Drain slopes (>0.4%) are necessary to overcome these adhesion forces by tractive forces in partially filled drain pipes. The inlet openings should be as lar as possible (>1.2 mm).

1. Introduction

Drainage is one of the most important methods of soil reclamation and improvement. For these purposes high investments are necessary. High investments are only justified if a long-term efficiency is guaranteed. This depends on research of wet sites, design of drainage systems, materials of pipes and filters, installation methods and maintenance of water levels in the main water courses.

Within the last twenty years much progress has been made in pre-drainage investigations of soils and in developments of new materials and installation techniques. Marginal sites, which had been excluded from drainage in former times, can now be reclaimed. Their drainage systems, however, need more maintenance.

Silting causes a widespread damage in drain systems. According to Henning (1966) three forms of disturbances of drainage discharge exist: silting (66%), ochering (30%), other reasons (rooting, subsidence, pipe-defects)(4%). For peat soils and mineral soils we get a different picture, namely:

soil type	silting	ochering	others
peat soils	43	56	1 %
mineral soils	79	15	6 %

In most cases silting and ochering are found together. It then becomes very difficult to restore the discharge function of a drain, because ochre and mineral particles can form a cement-like substance.

The aims of this study were:

- to determine the causes of ochering and the sites where it may be expected,
- to find methods of preventing ochering and
- to conduct experiments with equipment for cleaning iron-clogged and silted drain pipes.

Ochre is a colloidal to amorphous sludge which is generally rich in iron but may also contain Al, Mn, Ca, Mg, SO_4, and particularly C. It is deposited in wells, ditches and pipe drains, and reduces their discharge capacity (Table 1).

2. Types of ochre formation

This study is mainly concerned with the clogging effects of ochre in filters and drain pipes. Two types of ochre formation are distinguished:

- chemical ochre formation by self-oxidation and isoelectric precipitation of the dissolved Fe^{++}, because of changes in the pH-value (>pH 7.6) and the redox potential E_h (>300 m V),
- biochemical ochre formation by numerous autotrophous and heterotrophous iron-organisms.

In most cases the two processes occur simultaneously or in succession. The composition, consistency and adhesion of the ochre at boundary layers depend on the dominating type of ochre formation.

The water-dissolved iron pH < 4 originates from the weathering of rocks or from soil-forming processes (Tab.2). Generally the Fe^{++} content increases with the clay content. It decreases with increasing pH and E_h values of the soil suspension. Iron clogging may be permanent - allochthonic (of alien origin) or temporary - autochthonic (indigenous), the latter for example occurring in marshy moor, moor gley, or clay moor. Allochthonic iron clogging may even occur in clayless soils, if Fe^{++} is conveyed by groundwater from an extensive iron-rich catchment area. It is found particularly at valley margins in the presence of pressure or alien inflowing water. Autochthonic ochre habitats cause iron clogging only temporarily after drainage; its extent depends on the iron content of the soil and the intensity of aeration. Allochthonic iron clogging in drains is a permanent process, even if the iron content in the groundwater is low.

An orohydrographic analysis of the site, on the basis of geological-pedological maps, facilitates the forecast of the type of ochre formation. Certain site characteristics, such as colour of the surface water and of the soil profile, indicate the likelihood of ochre formation (Table 3). This first guess may be supported by analysis of water and soil samples for their total Fe, Fe^{++} and pH-values. The degree to which iron clogging is likely to occur can be determined by interpretation of our numerous comparative tests (Tables 4 and 5).

3. Degree of iron clogging

In recent years colorimetric field or laboratory tests were standardized to determine the iron content in water or soil. If one's aim is to protect drains from ochre, such tests can indicate the degree and type of ochre formation in each individual case, one must then decide whether:

- a pipe drain is not feasible because it will soon clog (allochthonic ochre, >1 ppm Fe in water, pH<7)
- ochre formation can be prevented (autochthonic ochre) or limited.

4. Counter-measures

There are three possible counter-measures:

a) Ochre formation should take place in the soil *before* the water enters the drain. Generally, this should be the aim if the ochre formation is autochthonic-transitory.

b) The ochre should precipitate in the ditch *after* the water has passed through soil, filter and drain pipe. This solution is particularly recommended if there is risk that the ochre formation will be allochthonic-permanent.

c) The third possibility is drain pipe maintenance by various, repeated cleaning methods. If neither of the first two methods can be applied because of soil technological, hydraulic engineering, and economic reasons, this will be the only solution.

The Society for Rural Water Resources Engineering (Kuratorium für Wasser und Kulturbauwesen KWK) supported 20 years of research programmes to test various ways of preventing, reducing and removing ochre deposition in drain pipes. The various problems, and their possible solutions obtained from different pilot surveys, are summarized as follows.

4.1 Ochre formation before the water enters the drain

4.1.1 Subsoiling

Subsoil loosening (repeatedly) promotes oxidation and precipitation of the iron in the soil, provided soil texture and moisture content (< plastic limit) are suitable. Subsoiling will only increase aeration if coarse pores (>10 µm) are created and maintained. Mole drainage may also contribute to such aeration effects. After immature, reduced soils with autochthonic ochre hazards are browned sufficiently, pipe drains can be placed in the pre-aerated soil.

However, to prevent a remobilization of Fe^{++}, any subsequent iron-reduction in the soil must be avoided.

4.1.2 Liming

The intensity of Fe^{++} oxidation in the soil depends not only on the availibility of oxygen, but also on the pH-value. The liming requirements of the soil increases with the clay content and decreases with the humus content (Table 6). Often line applications for ochre precipitation in the soil are higher than those needed for optimum fertility. A four-fold lime application in addition to drainage, for example, was necessary to precipitate 730 t Fe/ha in an alluvial meadow soil (Table 7). Recently oxidized (i.e. non-chrystalline) iron in the soil is liable to reduction if the pH-value and soil aeration decrease in the course of land use. Therefore, a relatively high Ca-level must be maintained in the soil by successive liming and it must be controlled by regular tests. Ca-losses via drainage may reach up to 0.8 t/ha in humid climates, whereas the withdrawal by crops is relatively small.

Liming of the whole field is often very expensive. Therefore, at another trial, liming was concentrated on the drain ditch or its immediate neighbourhood. The lime quantity (0.5 - 2.0 kg/m drain ditch) was thus reduced to 10% of the area application. This lime distribution forces most of the iron precipitation to occur near the drain, thus deteriorating the physical characteristics of the soil in this important reach. The permeability of limed drain ditches decreases substantially with increasing iron deposition (Table 8).

4.1.3 Filter materials

Coarse filter material in the refilled drain excavation (gravel, slag, or voluminous, coarse-pored filter jackets) creates an additional aeration space around the drain in periods when the groundwater level is below drain depth. Predominant ochre precipitation in the filter provides a clean drain, but it may clog the effective filter pores (Table 9).

4.1.4. Vyredox-Method

A final evaluation of the Vyredox-method is not yet possible because testing is still in progress. This technology was developed for water supply plants.

Oxygen-rich water is pressed via feeding wells into the groundwater around well galleries to promote iron oxidation in the soil. The inflowing external water disposes of most of its iron content in such iron zones by adsorption. Because of its high cost, this method is feasible only for localized, Fe-rich groundwater sources or for protection of important outlet ditches.

4.2 Precipitation of ochre in the ditch

The possibilities of promoting iron oxidation in the soil are restricted to certain sites (suitable for liming or subsoiling) and to the autochthonic ochre type. Therefore, the counter-measure 2 (iron clogging in the ditch) must be attempted in most cases.

4.2.1 Drain slope

A steep (artificial) drain slope is often considered a possible solution. The iron content of the drain water increases with the distance of flow and the chance for aeration towards the drain outlet, irrespective of the slope. The critical limit for the tractive force may be assumed only above 0.4% slope, in partially filled clay pipes of 50 mm ∅, which corresponds with the findings of other authors. As a matter of fact, iron clogging and sand deposition often go hand in hand. Soil particles >60 µm are increasingly detained with higher iron content in the drain sludge, while these may be washed out from iron free sediments (Fig.1).

4.2.2 Plastic pipes

The higher adhesion of ochre to plastic material must be considered even in smooth pipes. The adhesion to plastic surfaces exceeds that to clay pipes by up to 30% (Table 10). One of the reasons for this is the reaction to necessary production agents (stabilisators, emulgators). Thus a reduction of ochre adhesion by modified plastic material appears to be limited. This additional adhesion can be overcome only above 10 - 20% slope, depending on size and degree of partial fill of the pipes. Such high slopes are not available for drains (Table 11).

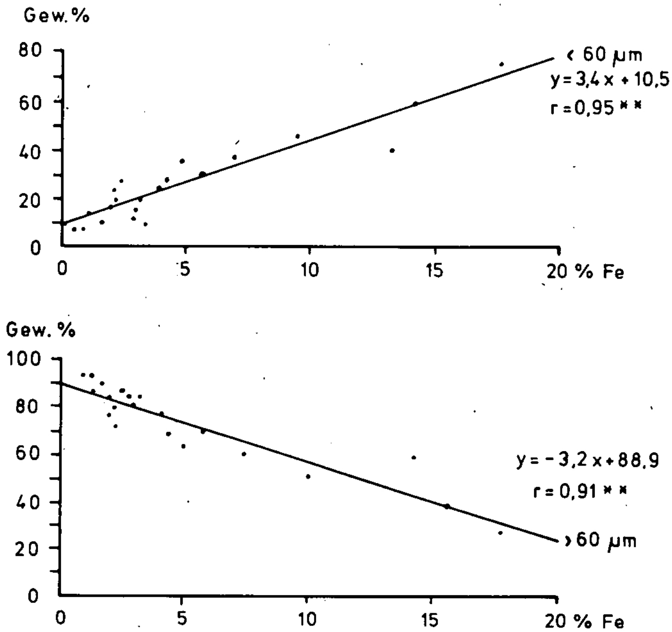

Fig.1. Content of iron and particles < 60 µm and > 60 µm in the drain sludge.

4.2.3 Inlet openings

The inlet openings into drains are particularly liable to iron clogging. Their size, form, and distribution must be carefully considered. On the one hand, they should be as large as possible to avoid hanging menisci between opening and surrounding soil particles. On the other hand, wide slots in texture-labile soil require jacket filters, which themselves are liable to iron clogging. If there is an ochre hazard the minimum opening width should be 1.2 mm.

4.2.4 Submerged drain outlets

Submerged drain outlets will prevent aeration of the drainage water, particularly in permanently water-filled drain systems. Special drain outlets, which back up water into drain pipes by syphon devices were, tested. This method is possible in very deep drains or section-wise in less deep, small systems. The method is restricted to very permeable soils because backing up reduces the tractive force of the water in the drain. Moreover, a mechanical filter is required (Tables 12 and 13).

4.2.5 Antoc filter

The application of partially decomposed organic filter materials (straw, chaff, sawdust) in soils with ochre hazard often results in the discharge of a grey-blue suspension instead of the yellow-brown drain-sludge. The grey-blue colour indicates an iron reduction and iron complex formation. Disintegrating organic material liberates chelatizing phenoles, like tannin (Table 14). Two particularly tannin-rich natural materials are the wood of the mimosa-bush (Accacia decurrens) and the fruit husk of the caucasian oak (Quercus valonea). These were embedded in straw and sold under the name Antoc-filter. This filter has a good ochre restricting effect for 2-4 years after drain construction (Table 15). However, the applied dosage (150 g mimosa/m) and the mostly too fine shredding caused a high extraction of the tanning shortly after completion of the drainage systems, which led to excessive oxygen consumption and fish-toxic effects (Fig.2). Therefore, a dilution of drain to surface water of more than 1:20 must be guaranteed. Antoc-filters, which supply Fe-complexes according to the iron content of the groundwater, are still being developed. Selective Fe^{++}-ion exchanges in filters are possible, but too costly up to now. Drain filters soaked with bactericides have not successfully stopped biological ochre formation, as the bactericides are extracted too soon. However, Cu-slag as slow, long lasting source for Cu^{++}-ions has substantially reduced microbiological ochre formation, particularly for filamentous iron bacteria.

4.3 Cleaning methods

4.3.1 Mechanical methods

All the above-mentioned measures require some additional effort, which counteracts mainly the initial pronounced ochre occurrence. In the long run continuous maintenance cannot be avoided to meet major allochthonic ochre hazards. A low pressure flushing with vacuum tank has proved successful for small systems with pure ochre deposits which are not yet consolidated by age.

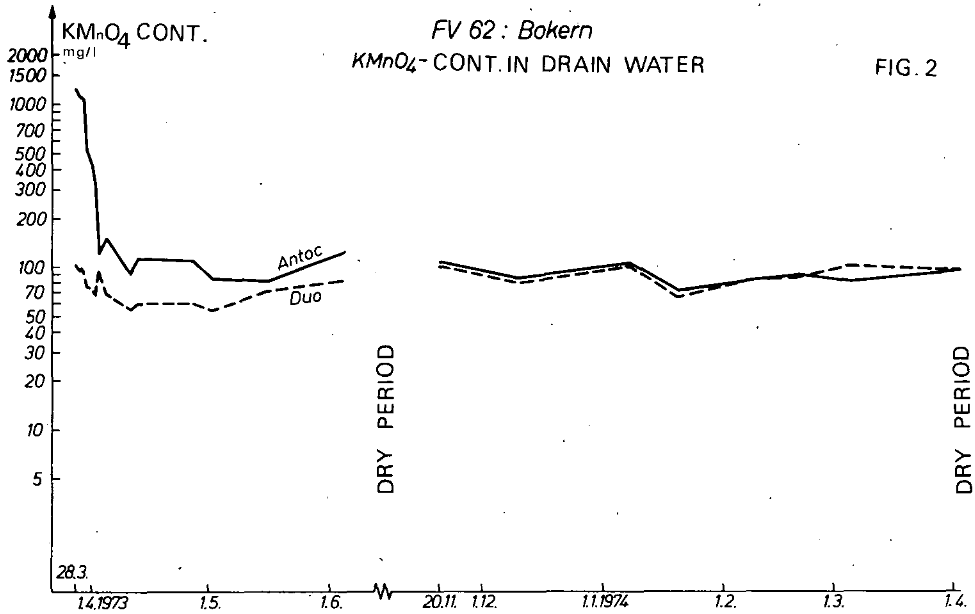

Fig. 2. COD of the runoff iron drains with Antoc and duo-filter (Scheffer and Kuntze, 1975).

Sand-ochre deposits are best removed by high pressure flushing. Individual drain pipes and open collector ditches are most suited for this method, as well as for all other functional controls. The expense of high pressure flushing depends on the degree of iron clogging, and on slope and diameter of the drain pipe, but less on the type of pipe (Table 16). Regular flushing every 2 - 3 years will maintain an ochre-endangered drain in good condition. The intervals between flushes should be shorter in the beginning.

4.3.2 Chemical methods

Very persistent aged ochre needs a radical cure with acids and reduction agents. In such cases the necessary acid-ochre proportion must be determined first, to avoid inflow of excess acid into the open ditches (Table 17). The best method is: to dam the acid in the drain for several hours, repump the acid ochre suspension into tank-wagons, and repeat the process with fresh acid. A subsequent mechanical water flushing is generally required in addition.

5. Final remarks

The last example demonstrates that a successful protection against ochre requires a combination of different measures. The efficiency of the methods mentioned depends on a timely recognition of ochre type, the degree of clogging, and its aereal extent.

Ochre-endangered sites always need drainage. Generally they have a good soil permeability and drainage is possible. However, it depends on the potential to avoid iron clogging by counter-measures with lasting effects whether drainage of such sites is feasible.

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TABLE 1. Composition of iron ochre from drain pipes (n = 21)*

Fe_2O_3	3.0 - 65.9
Al_2O_3	0.2 - 39.3
MnO	0 - 12.9
CaO	0.06 - 12.8
MgO	0.13 - 1.55
SO_4^{--}	0 - 7.54
CO_2	0 - 9.04
Org. matter	25.5 - 53.5
C/N	11 - 30
pH	3.6 - 7.1

**in % of dry matter without solid fraction (L.Petersen, 1966)*

TABLE 2. Iron content of rocks, sediments, soils and clay minerals*

Mineral	Fe_2O_3	FeO
Magmatite		
Granite	1.6	1.8
Syenite	2.7	3.3
Diorite	3.2	4.4
Gabbro	3.2	6.0
Basalt	5.4	6.4
Sediments		
upper Bunter (clay)	4.4	-
lower Bunter (sand)	3.6	0.1
lower Muschelkalk (limestone)	0.7	-
loess	1.8	0.4
wind blown sand	0.2	-
Soils (acc. clay content)	0.1 - 5	
Clay minerals		
Kaolinite	-	
Smectite	0 - 30	
Illite	2 - 5	
Vermiculite	3 - 12	
Chlorite	0 - 15	
Iron containing minerals in soils		
Siderite		$FeCO_3$
Pyrrholin		FeS
Pyrite		FeS_2
Vivianite		$Fe_3(PO_4)_2.8H_2O$
Goethite, Limonite		$\alpha FeO(OH)$
Lepidokrokite		$\gamma FeO(OH)$
Haematite		αFe_2O_3
Maghemite		γFe_2O_3
Magnetite		Fe_3O_4
Jarosite, cat clay		$K_2Fe_6(OH)_{12}(SO_4)_4$

** acc. Scheffer and Schachtschabel, 1976*

Table 3. Characteristics of sites for estimating iron ochre damage•

Danger of iron clogging	Surface	Colour of water	Bottom	Soil
very low	free	clear colourless	free of ochre	equal, diffusse iron colour without oxidation and reduction horizons
low	sporadic oily	some ochre flakes	thin layer of ochre flakes	sporadic iron stains
medium	wide spread oily	many ochre flakes	red-brown coloured sediment	iron stains
heavy	coherent oily	light brown-red colour	distinct ochre sediment	iron concretions
very heavy	coherent oily	brown-red colour muddy	heavy layer of ochre sludge	bog iron-ore distinct oxidation & reduction horizons

• *Kuntze and Eggelsman, 1968*

TABLE 4. Iron ochre damaging for tile drains depending on Fe^{++}-content and pH of the groundwater•

Fe^{++}-content (mg/l)		Degree of damaging
ph >7	pH>7	
<0.5	<1.0	none
0.5 - 1.0	1.0 - 3.0	small
1.0 - 3.0	3.0 - 6.0	medium
3.0 - 6.0	6.0 - 9.0	high
>6.0	>9.0	very high

• *Kuntze and Eggelsman, 1968*

TABLE 5. Delimination of the soils by the danger of iron ochring from the Fe^{++}-content and pH of the groundwater•

Soil (n=)	Groundwater		Danger of iron clogging
	Fe^{++} (mg/1)	pH	
high bog (8)	0.5 (0.3 - 0.7)	4.5 (3.5 - 5.5)	none
low moor (18)	5.4 (0.2 - 33.5)	6.0 (4.7 - 7.4)	low - medium
transition moor (15)	8.7 (1.8 - 19.3)	4.8 (3.6 - 6.1)	medium - high
peaty soil (31)	8.7 (0.3 - 43.3)	6.2 (4.3 - 7.0)	low - high
clay pan moor (20)	13.8 (2.1 - 58.5)	6.3 (4.3 - 7.7)	high - very high

• *Kuntze, 1966*

TABLE 6. pH levels of soils needed to prevent deposition of ochre in drains

% 2 μ	soil**	4(h)	4-8($\bar{h}$)	8-15($\bar{h}$)	15-30(a)	30-60(H)	> 60(H)
0 - 5	S	5.5	5.0	4.5	4.5	4.0	4.0
5 - 12	lS	6.0	5.5	5.0	4.5	4.0	4.0
12 - 17	sL	6.5	6.0	5.5	5.0	4.5	4.0
17 - 25	L	7.0*	6.5	6.0	5.5	4.5	4.0
> 25	L, LT, T	7.0*	7.0	6.0	5.5	4.5	4.0

* favourable, if additional free $CaCO_3$ well distributed in the soil, is present for stabilization of soil structure

** S = sand sL = sandy loam LT = loamy clay
L = loam T = clay LS = loamy sand

TABLE 7. kg/ha Fe-total precipitated in the soil (0-140 cm) with different drainage and liming*

Undrained	Drained + CaO	Drained + Ca1	Drained + Ca2	Drained + Ca4
209 377	354 187	460 590	467 564	730 047 kg/ha. compared with undrained
	+144 810	+106 403	+113 337	+375 860 kg/ha compared with unlimed but drained

* *Kuntze, 1967*

TABLE 8. Physical and chemical qualities of soils in the drain trench (1966)*

cm under soil surface	Permeability (cm/day)			Total pore spaces (vol.%)		
	a	b	c	a	b	c
35 - 40	29.9	1.5		47.5	42.5	
60 - 65	20.1	8.6	10.9	47.6	55.2	46.5
100 - 105			11.6			49.4
	vol.% macropores (>50μ)			vol.% macropores (50-10μ)		
35 - 40	11.2	3.9		4.1	2.5	
60 - 65	8.4	6.6	9.6	2.5	1.7	3.2
100 - 105			3.1			3.7
	%Fe - total			mg Fe^{++}/100 g soil		
35 - 40	3.20	8.07		18	6	
60 - 65	3.89	3.11	2.48	29	7	4
100 - 105			3.79			138
	% CaO			% $CaCO_3$		
35 - 40	0.14	0.42		-	0.89	
60 - 65	0.14	0.33	1.55	-	-	1.08
100 - 105						

* *a) unlimed*
b) refilled soil mixed with lime (25 kg/m³ - 1956)
c) refilled soil mixed with lime (100 kg/m³ - 1956)

TABLE 9. Permeability of drain filter materials 4-12 years after installation *

Filter material	inch/hour
sawdust, coarse	15.41
fine sand	8.92
humic crumb soil, fine sand	5.14
sawdust, decomposed	0.40
slag with supporting $Fe(OH)_3$	0.25
sawdust with FeS	0.03
humic crumb soil, fine sand with FeS	0.03
glass fibre envelope with $Fe(OH)_3$	0.02

Ford et al., 1968

Table 10. Adhesion (g/cm^2) of different suspensions at surfaces of materials (brick = 100) *

Material	Water	Artificial ochre	Bentonite	Bentonite/ochre	Natural ochre
brick	0.315±0.003(100)	0.423±0.002(100)	0.405±0.002(100)	0.401±0.004(100)	0.369±0.003(100)
ultramid	0.395±0.004(126)	0.442±0.002(104)	0.429±0.004(106)	0.437±0.002(109)	0.436±0.003(118)
PE-hard	0.395±0.004(126)	0.482±0.002(114)	0.475±0.002(117)	0.478±0.004(119)	0.434±0.003(117)
PVC-hard	0.370±0.006(117)	0.513±0.003(121)	0.481±0.001(119)	0.475±0.002(118)	0.444±0.002(120)
PVC-firmed	0.405±0.003(129)	0.529±0.003(125)	0.503±0.003(124)	0.500±0.001(125)	0.453±0.002(123)
Polystyrol	0.390±0.005(124)	0.556±0.002(131)	0.517±0.003(128)	0.528±0.002(132)	0.493±0.002(134)

* *Kuntze, 1968*

TABLE 11. Tractive force of water (g/cm^2) within drain pipes of different diameters depending on slope and degree of filling *

Filling of pipe	% slope of pipe						
	0.2	0.5	1	2	5	10	20
				diameter NW 50 mm			
0.1 d	0.0006	0.0015	0.0030	0.006	0.015	0.030	0.06
0.2 d	0.0012	0.0030	0.0060	0.012	0.030	0.060	0.12
0.4 d	0.0021	0.0054	0.0107	0.021	0.054	0.107	0.21
1.0 d	0.0025	0.0063	0.0125	0.025	0.063	0.125	0.25
				diameter NW 100 mm			
0.1 d	0.0012	0.0030	0.006	0.012	0.030	0.06	0.12
0.2 d	0.0024	0.0060	0.012	0.024	0.060	0.12	0.24
0.4 d	0.0043	0.0107	0.021	0.043	0.107	0.21	0.43
1.0 d	0.0050	0.0125	0.025	0.050	0.125	0.25	0.50
				diameter NW 160 mm			
0.1 d	0.002	0.005	0.010	0.02	0.05	0.10	0.20
0.2 d	0.004	0.010	0.020	0.04	0.10	0.20	0.40
0.4 d	0.007	0.017	0.034	0.07	0.17	0.34	0.68
1.0 d	0.008	0.020	0.040	0.08	0.20	0.40	0.80

* *Kuntze and Eggelsman, 1973*

TABLE 12. Iron ochre in aerated and unaerated pipes*

Place of sampling in the drain pipe	Branch drain with free outlet	Branch drain with back water outlet
upper third	2.75	2.12
middle third	2.23	0.57
lower third	1.06	0.87

* *Kuntze, 1968*

TABLE 13. Degree of ochring of drains with free outlet and backwater*

Place of sampling in the drain	Branch drain with free outlet		Branch drain with backwater outlet	
	g/m	vol. %	g/m	vol. %
upper third	177.3	7.2	177.7	7.4
middle third	183.7	7.7	136.2	5.7
lower third	68.2	2.9	336.5	14.0
average	176.7	7.4	233.4	9.7

* *Kuntze, 1968*

TABLE 14. Content of phenols and $KMnO_4$ of extracts of filter materials with tannin (in g of dry matter)*

Material	Phenols	$KMnO_4$
high moor peat, slightly decomposed	0.43 mg	59.3 mg
wood wool	1.02 mg	73.5 mg
rye straw	0.37 mg	95.3 mg
spruce sawdust	0.05 mg	49.8 mg
oak sawdust	1.86 mg	110.6 mg
Mimosa[1]	51.3 mg	1990.0 mg
Trillo[2]	53.9 mg	2746.0 mg

* *Kuntze and Scheffer, 1974*

[1] *Wood of the acacia decurreus or A. molissima*
[2] *Fruit of the Quercus valonea*

TABLE 15: Average silting and ochring degree depending on filter material*

Filter	Date	Dry matter (g/m)	HCl-insoluble (g/m)	Fe_t (g/m)
Duo	IX/74	67.7	1.9	13.8
Antoc	IX/74	16.1	1.0	2.5
Duo	X/76	29.7	4.8	6.0
Antoc	X/76	31.9	7.8	5.0

* *Ochre trial Bokkern, installed 1972*

TABLE 16. Demand of time and water consumption (l/50 m) with high pressure cleaning of ochred drain pipes

Diameter			Type of pipe			Slope			Degree of ochring		
(cm)	(a)	(b)		(a)	(b)	(%)	(a)	(b)	(g/m)	(a)	(b)
4	3'	93	clay	2'54"	118	0.25	2'42"	107	<. 100	2'42"	98
5	2'48"	111	PVC	3'30"	107	0.35	3'30"	116	100-250	3'12"	117
6.5	2'42"	110	PE	2'30"	93				> 250	7'	160
7	4'	120									

a = demand of time *b = water consumption*

TABLE 17. Chemical dissolving of ochre after 1 hour

Solution		pH	Acid:ochre	Undissolved percentage of iron
HCl	10%	0	62:1	0
HCl	10%	0	23:1	0
HCl	10%	0.2	13:1	0
HCl	1%	1.1	62:1	62
HCl	1%	1.4	23:1	88
HCl	1%	1.8	13:1	95
H_2SO_3	7%	1.2	62:1	0
H_2SO_3	7%	1.4	29:1	36
H_2SO_3	7%	1.8	13:1	57

Group 300

Installation methods

CONTENTS

Paper 3.01

TECHNICAL IMPROVEMENTS DESIRED IN SWEDISH LAND DRAINAGE

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Summary

Today's drainage techniques and desired improvements are discussed.

1. Introduction

Swedish agriculture contains about 3 000 000 hectares of cultivated land distributed amongst about 100 000 holdings. From an investigation by "The National Central Bureau of Statistics" in 1974 we know that 1 018 500 ha are systematically drained while 1 950 500 ha are not. The investigation also says that 300 000 ha was drained already before 1930 and that 618 500 ha require drainage today (for the first time or anew). We estimate that about 1 300 000 ha can produce successfully without or with sporadical drains.

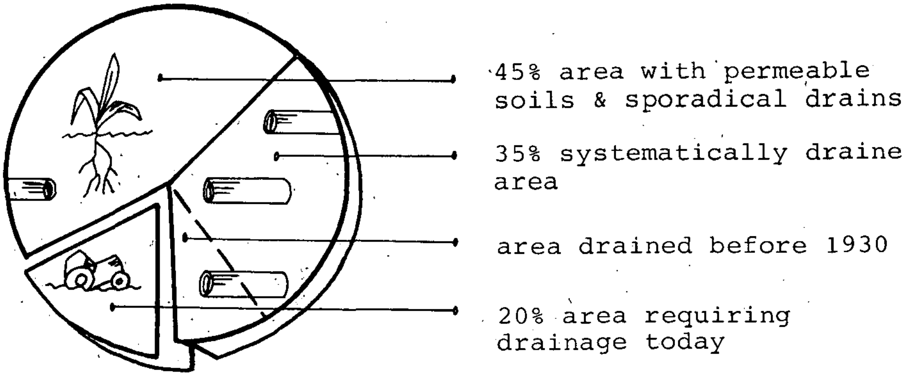

Fig.1. Distribution of Swedens 3 million ha cultivated land. (Boundary lines are naturally not sharp.)

Yearly 15 000 - 19 000 ha are drained in Sweden. We consider that this is inadequate and we often discuss the need of some kind of stimulus to these kinds of long lasting investments.

Projecting is not compulsory but almost every hectare is still projected by one of the 24 County Boards of Agriculture. Projecting fee is about 150 Skr/ha* which means 3-4% of the total land drainage cost. The average size of such a plan is nowadays 12,5 ha embracing about 6 700 m mains and laterals.

Drainage costs have developed during the last years almost exactly as the general inflation in Sweden, which means that land drainage is still as profitable as before. Of course there is a great cost difference between drainage performed with direct-laying special-machines (chain- or weeldiggers) and the more expensive method with excavator and manual pipelaying. Average costs are now about 5 000 Skr/ha for performance and material, including laterals and main pipes up to about 200 mm diameter. (We often use main pipes bigger than 200 mm but in cost statistics we call those pipes culverts and not land drains.)

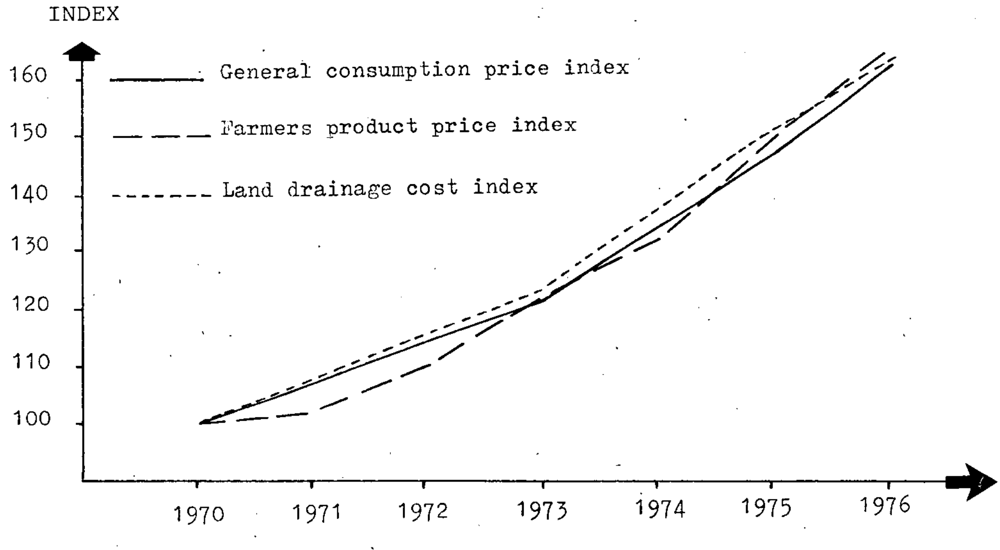

Fig. 2. Drainage costs in Sweden develop almost exactly the same way as the general inflation.

** 100 Skr = 21,2 USD = 48,8 NLG = 45,5 DEM = 11,1 GBP*

Pipe material for land drainage was previously tiles from several provincial brickyards, but today corrugated PVC-pipes dominate clearly. This change has had a strong influence on the drainage technique and on saving of labour. Three years ago we got a Swedish (and Scandinavian) standard for manufacturing and testing PVC- and PEH-pipes. Swedish standard 3063, 3064.

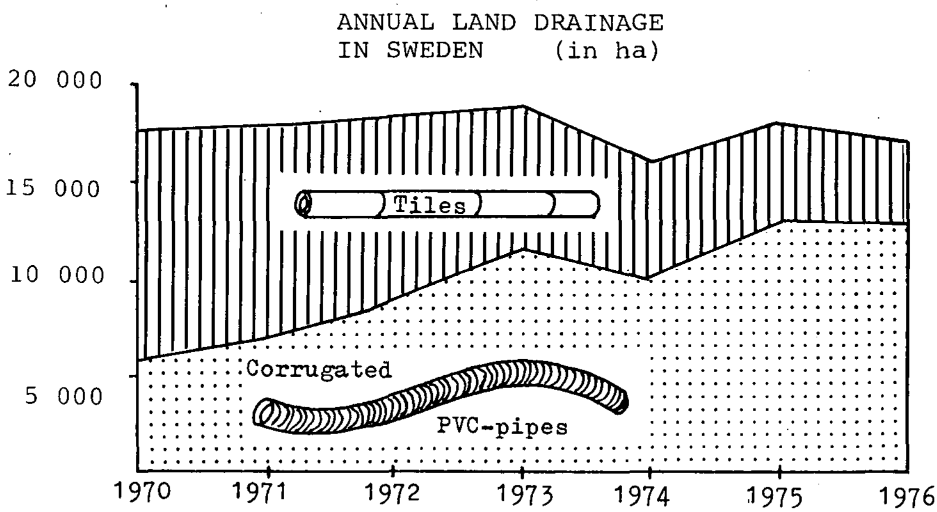

Fig.3. During the last five years Swedish land draining has changed pipe material from tiles to corrugated PVC.

2. Today's drainage technique

Land drainage in Sweden is always performed by special contractors. They are organized in small companies with one or a few machines each and two or three workmen on each machine.

About 80% of the land drainage can be done with special machines of different wheel- or chain types. The remaining 20% of the soils are so stony that we have to use common excavators and manual pipe laying. There is still no trenchless machine in use, except for cable works.

Most laterals have 50 mm inner diameter and they are layed down 80-100 cm deep; 120-140 cm in organic soils. Lateral spacings vary from 12 to 20 m in moraines and mineral sediments, and from 15 to 40 m in organic and muddy soils. Lateral spacing is determined mainly depending on soil permeability and trafficability demands.

Nowadays no drainage machine is manufactured in Sweden. From Finland we import a digging-wheel-machine UKKO-MARA built on a Ford 7000/5000 frame. On the same frame this factory also builds a chain-digging-machine, MARA 655. From England we import the Dutch designed BARTH K 140 and K 150, both chain digging machines with caterpillar frames. Recently one K 170 was delivered to SW-Sweden, thus indicating the contractors interest in increasing his capacity and running reliability. From England also comes a HOWARD MK V, and a Swedish designed DRAINER, both chain diggers; and from Norway a digging-wheel-machine, RAADAHL, built on a Muir-Hill-frame.

DIGGING TIME (in %) OF THE TOTAL MACHINE WORKING TIME
IN THE FIELD

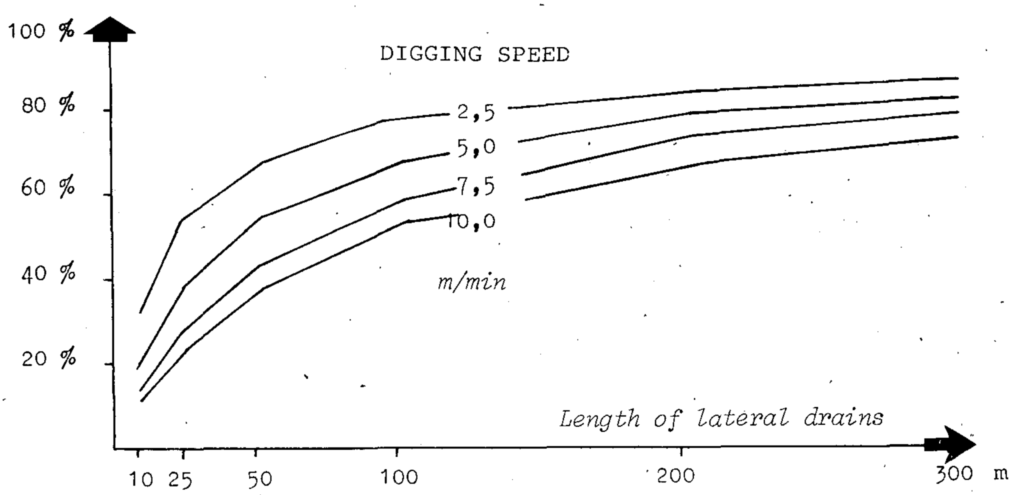

Fig.4. Digging time in percentage of the total machine-working time in the field depends on digging speed (2.5-5.0-7.5-10.0 m/min) and on average length of lateral drains (10-25-50-100-125-200-300 m). Field study from county Skaraborg in SW Sweden.

In one year Swedish contractors buy all together 8-10 new machines, mainly Finnish MARA and Dutch BARTH. These moderate size machines have shown to be rather suitable for Swedish conditions where digging depth normally is varying between 0.80 and 1.25 m, and where the drainage projects are not bigger than a few thousand metres each. Normally the machine must be transported on the roads between different projects once or twice a week.

For topographical reasons the average length of lateral drains is limited to about 100 m in Swedish land drainage. Therefore, the machines must move quite often from one position to the next. Danish and Swedish field investigations show that the length of laterals has more economical importance the higher digging capacity the machine has. A high-capacity machine is more expensive and must therefore be used more effectively.

3. Technical improvements

It is very expensive today to develop a new drainage machine and therefore design and performance must be adapted more to an international market rather than to needs and desires in different countries. Following desires are mentioned at first hand with reference to aims and conditions in Swedish land drainage.

> The draining season is always limited by our winter climate. The contractors are urgent to start in early spring (March-April) and continue till late autumn (November-December). The limited soil trafficability in spring and autumn leads to a desire that ground pressure of the drainage machine must be limited and preferably not bigger than 30 kPa/cm^2. Many machines already realize that demand while there are still improvements to do on supplying machines such as gravel hoppers, etc. Our limited digging season also brings about a desire that the running reliability of the machine should be high enough to permit hard work with little demand of repairings.
>
> The machine driver is expected to perform a careful output of work 8-10 hours per day. It is therefore necessary to offer him good working surroundings and from this point of view many desires can be drawn up. Permanent vibrations can be irritating and even dangerous to one's health. Cold (or hot) weather can be very trying if the machine does not have a protecting cab. Many machines only have manual depth regulation. It is in that case to be wished that the sighting line can be set up in a comfortable position and not backwards as to be found on some machines.

- To get maximum capacity from the projected pipe dimension and to get maximum length of the drains in hilly fields it is necessary to follow the projected depth and gradient very carefully, with a fast-digging machine that claims some kind of automatic depth control. Many different systems have been tested. Experience has shown that the rotating laser beam gives very good digging results with high security. This system is therefore superior to but also much more expensive than other systems. However, with a well-working laser system it is proved that a three-man contractor team can be reduced to a two-man team without capacity losses if you use chain- or wheel diggers with direct-laying of corrugated PVC- or PEH-pipes. On the other hand, if you already have a two-man team you can increase the digging capacity by about 25% or 300 m per day when using automatic laser-plane control. Thus an expensive equipment can still be profitable, and also give a better result.
- In Swedish soils it is very urgent to arrange a permeable backfill around the pipe. Normally gravel or coarse sawdust to a level of 2-10 cm above the pipe is used for that purpose. It is a desire, therefore, that the digging chain or wheel leaves a clean bottom of the ditch, also when the field is ploughed or harrowed. We have noticed that many imported machines for that purpose must be completed and reconstructed with soil-ploughs or similar equipment. It is a desire that this problem should be payed more attention to, already in design and manufacturing.

4. Trenchless draining

Trenchless draining is not yet tested in Sweden. A few contractors have shown some interest in buying such machines, but after discussions and investigations of the prospects they have declined.

We must remember that trenchless machines are very expensive and that the average size of our drainage projects is not bigger than about 5,000 m each. We also think that it can be difficult to supply the machine continuously with those quantities of filter material (over 1 m^3 per 100 m) we need. Finally, in certain regions re-draining already constitutes up to 50% of all draining projects. In re-draining it is often necessary to connect old drains with the new system and you can do that only when you know where the old drains are. We think that it might be difficult to observe those old drains with the trenchless technique, as well as to observe all different types of cables there are in an agriculture province.

We are, in spite of all desires mentioned here, still tempted to say that we are rather satisfied with the drainage technique we use today. We think that in the single case drainage results (effect, and length of life) today are more depending on the contractor team and the working conditions, than on the drainage machines.

Paper 3.02.

METHODS OF LAND DRAINAGE IN DENMARK

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Summary

The methods of land drainage, covering materials, and soil mechanics are discussed briefly below.

Introduction

The methods of land drainage in Denmark are not very advanced in the technical respect. During the last 10 years the trenching has been done mainly by means of different kinds of back-acting excavators. The subsequent installation of pipes and application of covering materials is done by hand. About 90 per cent of the drainage is carried out in this way (Table 1).

Approximately 4 per cent of the total drainage (5,000 ha/year), i.e. 200 ha, has been done by means of continuous excavating drainage machines with trenching chain during the last 2-3 years. Application of covering materials directly on the pipes is possible immediately after placing of the pipes in the ditch. This type of drainage machine performs a good quality of work with tile pipes and with plastic pipes as well, and should be an alternative in the future land drainage in Denmark. However, because of a very fluctuating rate of drainage, the contractors are not very happy to invest in such machinery without alternative utilization possibilities.

Another problem is the size of the different fields - many of the drained fields were only 1-3 ha, which is not sufficient to cover the cost of transportation of the machinery.

TABLE 1. DRAINAGE METHODS IN DENMARK
(rounded figures)

YEAR	AREA HA	TRENCHING, PCT. EXCAVATOR	CHAIN-TRENCHER	TRENCHLESS PCT.
1970	5000	94	6	0
1971	6000	96	4	0
1972	5000	95	5	0
1973	4000	84	7	9
1974	4500	86	7	7
1975	6000	87	4	9
1976	5500	93	3	4
1977	4500	86	4	10

The method of trenchless pipe drainage has been applied for 4-10 per cent of the total drainage during the last 5 years, mainly by use of unwrapped, corrugated plastic pipes. The result differs from place to place, but in larger fields with uncomplicated soils it seems to be a useful alternative because of the low costs.

Covering materials

Particularly in the western part of Denmark the soil conditions are very difficult in view of pipe drainage (post-glacial soil) and a very careful covering of the main part of the pipes is absolutely necessary.

Sawdust from coniferous trees is commonly used as covering material. The contents of resin have a preserving effect on the material. The sawdust, as covering material, has a double function, as it partly prevents fine particles to enter into the drain, and partly facilitates the flow of water into the drain (Table 2).

TABLE 2. CHARACTERISTICS OF COVERING AND PREWRAPPED DRAINAGE MATERIALS USED IN DENMARK

MATERIAL	FILTERING QUALITY	INCREASE IN PERMEABILITY
COVERING MATERIALS:		
TOP SOIL		+
SAW DUST	+	+
SPHAGNUM	+	+
SAND-GRAVEL	+	+
PREWRAPPED MATERIALS:		
SPHAGNUM	+	+
COCONUT FIBRE		+
STRAW		(+)
ANTOC		(+)
FIBERTEX	+	

Sphagnum, coconut fibre, and antoc are used only to a very small extent in practice

To some extent gravel is used as cover material, but compared with sawdust it is quite heavy, and the manual application into the trench is quite expensive. The cost of both materials is almost the same.

To prevent fine particles to enter into the drain from below, a tissue called "Fibertex", is often applied as a filter underneath the pipes, with for example sawdust covering the pipes. By this combination a very effective and durable filter is obtained, and the method is useful for drainage work by excavators with manual pipe installation and for the chain-trench machines as well, but not for trenchless drainage.

In certain areas there is a very heavy precipitation of iron compounds, ochre, in and around the drain pipes. This sometimes causes complete clogging of the pipes. In such areas always 8 or 10 cm pipes are used in the lateral drains. Besides tile pipes, plastic pipes with large perforations 2-3 mm are used.

Soil mechanics

By using back-acting excavators and to some extent chain-trench machines, the replacement of loose soil material increases the permeability of the soil around the pipe.

The method of trenchless drainage may cause some compaction of the soil around the drain. Experiments in Denmark have shown a lower drainage effect compared to the excavating method, especially when the drainage is carried out under wet soil conditions in loamy soil. However, the difference in effect seems to decrease after a few years.

Paper 3.03

GENERAL INFORMATION ON SUBSURFACE DRAINAGE IN THE NETHERLANDS

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Summary

A review is given on geographic conditions, the present state of drainage and the various projects in which drainage plays a role.

1. Introduction

This paper aims at providing some general information on subsurface drainage in the Netherlands for participants of the Drainage Workshop from abroad. It is also intended to supply some background information for the discussion notes of the Dutch participants, in which specific aspects of subsurface-drainage research will be considered.

The term subsurface-drainage means the lowering of groundwater level by means of horizontal, closed drainage systems. Other methods of drainage - for example (deep) draining by wells or drainage by ditches - will hardly be mentioned. Drainage of excess water to the sea by watercourses, streams, rivers and canals will not be considered, nor will water supply by means of infiltration.

On the whole, the subject is approached with agricultural applications in mind. Applications in horticulture - for example in greenhouses - will not be considered.

2. Conditions

2.1 Climate

The Netherlands lie in the temperate climate zone (51 to 53,5 ^{o}N) and have an oceanic climate. The daily air temperature averages are lowest in January (+1,7 ^{o}C) and highest in July (+17,0 ^{o}C).

The annual averages for the total amounts of precipitation and evaporation - measured on an area of open water - are 765 mm and 691 mm, respectively. The precipitation is rather evenly distributed throughout the year, while evaporation reaches a peak in the summer. The graph of Fig.1, which is drawn on the basis of a normal annual average, shows that in the period from April to August there is excess evaporation (203 mm in total) and in the period from September to March excess precipitation (277 mm in total).

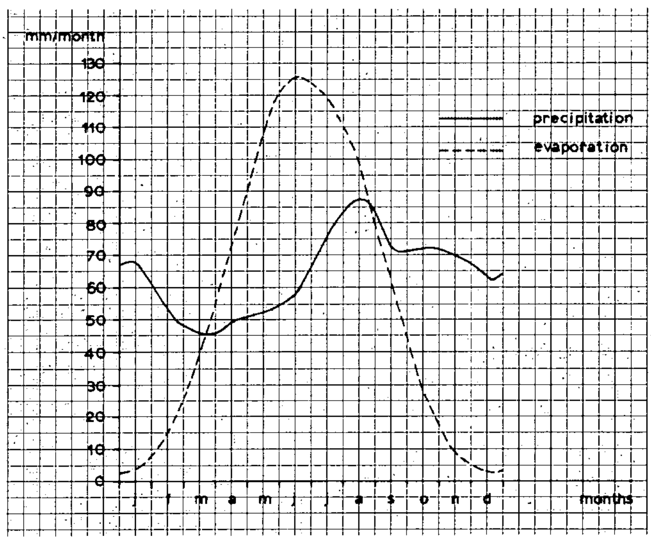

Fig.1. Trend of the average amount of precipitation and evaporation per month (De Bilt 1931-1960, KNMI).

2.2 Topography

The area of The Netherlands is about 41 000 km^2 (4 100 000 ha) of which 61 per cent, or about 25 000 km^2 (2 500 000 ha), is farmland. Of the total area about 27 per cent - protected by dikes and dunes - is below the average sea level; 71 per cent lies between 0 and 50 m above sea level, while only 2 per cent is higher than that. The lowest point, 7 m below sea level, is in the west of the country, and the highest point is 321 m above sea level in the extreme south.

Because the country is so low the groundwater level is usually found at a limited depth below ground level. Table 1 shows this.

TABLE 1. Percentages of the agricultural area in the different water-table classes

Water-table classes	Waterlevel in m below surface		%
	mean highest	mean lowest	
I	< 0,20	< 0,50	1
II	< 0,25	0,50 - 0,80	11
II	0,25 - 0,40	0,50 - 0,80	1
III	< 0,40	0,80 - 1,20	14
III	0,30 - 0,50	0,80 - 1,20	5
IV	0,40 - 0,80	0,80 - 1,20	5
V	< 0,40	> 1,20	18
V	0,30 - 0,50	> 1,20	9
VI	0,40 - 0,80	> 1,20	26
VII	> 0,80	> 1,60	8
VII	> 1,20	> 2,00	2

From this table it appears that the depth of the groundwater in the winter on 90% of the farmland is less than 1 meter. As a consequence of too high groundwater levels there is more than 10% yield reduction in at least 40% of the grassland area and 20% of the arable area.

TABLE 2. Average area of topographical plots

Classified plot area*	< 1,0 ha	1,0-1,5 ha	1,5-2,5 ha	2,5-4,0 ha	> 4,0 ha
Percentage	8	28	40	13	11

* *Plots are averaged per municipality*

This table shows that on the whole the plot size in the Netherlands is rather small; 76 per cent of the farmland is situated in municipalities with an average plot size of less than 2,5 ha. As the plot size in arable areas should be at least 7,5 ha and in grassland areas at least 3 ha, the existing plot size is unsatisfactory for good agricultural exploitation.

In view of the small plots and the number of ditches surrounding them, there are obviously a great many ditches in The Netherlands (Photo 1). The water levels in these ditches vary depending on the season and the level control in the areas concerned. Table 3 gives an impression of the water levels that often occur. The data in this Table are estimates and are not based on an accurate survey.

TABLE 3. Depth of ditch-water levels in m below ground level

Areas	in summer	in winter
clay soils	0,75 - 1,50	1,00 - 1,50
low-land peat soils	0,25 - 1,00	0,25 - 1,00
sandy soils	0,50 - 1,25*	0,50 - 1,25

* *Ditches and watercourses situated in sandy areas often contain no water in the summer, because the groundwater level has dropped below the bottom of the ditch with no possibility of a water supply.*

2.3. Soils

The soils in the Netherlands are rather varied as can be seen from the generalized soil map of the Netherlands on Fig. 2. Not all the features on this map are important for subsurface-drainage. The main ones can be summarized in the following four groups:

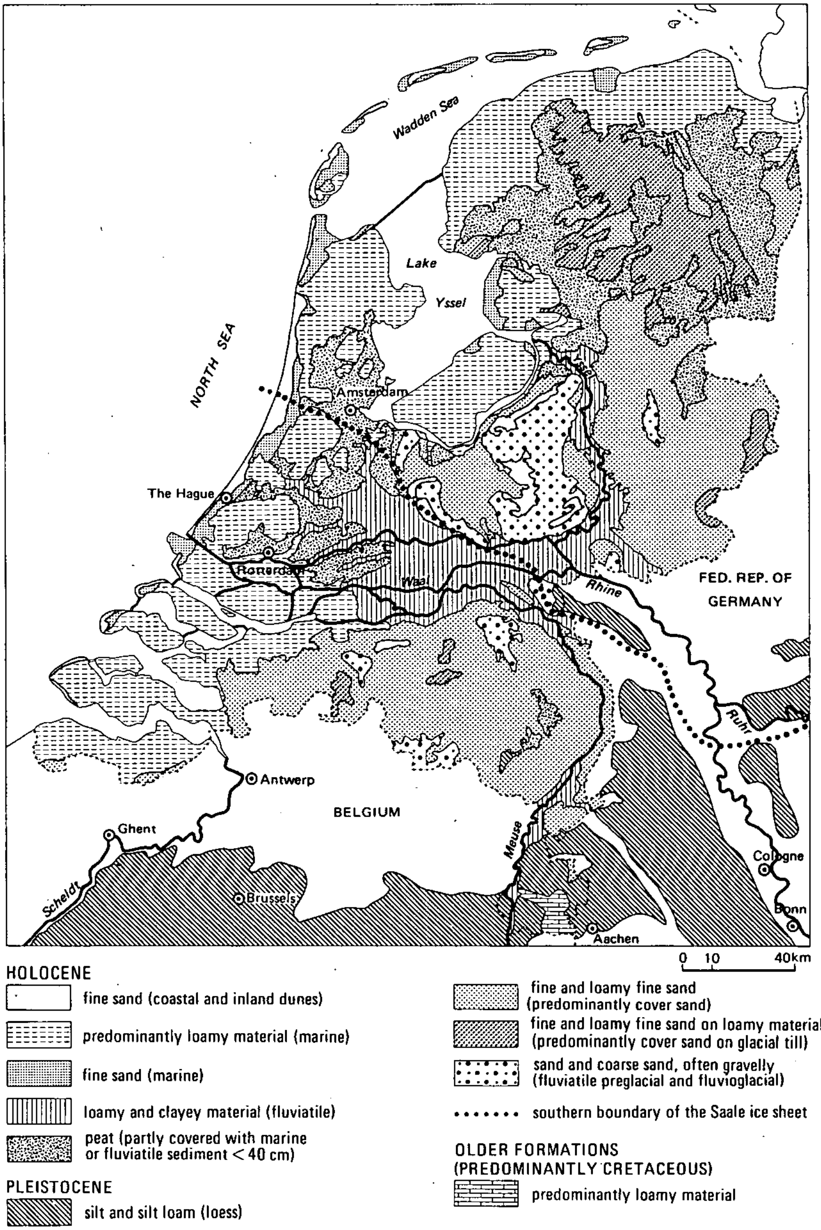

Fig.2. Parent material and surface geology in The Netherlands (De Bakker, 1978). In Belgium and the Federal Republic of Germany only the loess is indicated.

1. the marine loamy soils, in the north and south-west;
2. the loamy and clayey fluviatile soils, in the centre;
3. the low-land peat soils, in the west and north-west of the IJssel Lake;
4. the sandy soils in the east and south.

Before describing these 4 groups in more detail let us make the following comments.

a) The materials, in the top layer of mineral soils, have almost always been transported by wind, water or glaciers. Soil weathered from solid rock is only found in the south-east of the country. The material deposited by wind can be found in the Holocene coastal and inland dunes and in the Pleistocene sand and loess; the material deposited by water is either of Holocene marine origin or of Holocene fluviatile origin; the material of glacial origin is either glacial till or boulder clay, mainly found in the north of the country.

b) Nearly all the soils are easily workable; in other words, there is no serious mechanical resistance when digging the trenches. This is just as true for trenchless subsurface-drainage. One exception is the heavy river-clay soil (the clayey part of the fluviatile area) when it is extremely dry.

c) It has already been mentioned that many of the cultivated soils are too wet. All cultivated soils are, to a certain extent, supplied with a drainage system. This means that for many months of the year the area has sufficient bearing capacity for the mechanical installation of the drainage pipes. However arable areas in particular are not drained during the growing season. The danger exists, therefore, that the subsurface-drainage system is constructed at a time when the groundwater levels are too high, which can seriously affect the quality of the subsurface-drainage.

1. *Marine soils*

The clay content of these soils varies from 8 tot 40 per cent. The soil is often stratified and the clay content decreases with depth. Sea-clay soils are naturally calcareous but the older soils are usually only slightly calcareous in the top soil.
The permeability factor (K) of these soils is usually between 0,05 and 0,50 m/d. Greater permeability occurs, for example, in the Lake Yssel Polders (0,50 to 5,0 m/d), while there are lower permeabilities in the heavy non-calcareous clay soils. Most of the sea-clay soils are used for arable farming.

2. *Fluviatile soils*

These soils are usually calcareous and of loamy texture; the deeper subsoils are coarse in texture, as far as they are situated on the natural banks of the rivers. These soils are about 1 m higher than the general level of the floodplain. Because of this relative elevation and permeability of the subsoil, these soils need hardly any subsurface-drainage.

Soils situated in the backswamps are about 0.5 m below the general level of the floodplain. These soils are always non-calcareous and clayey, both in the topsoil and in the subsoil. The subsoils are cracked locally and then have a high permeability (1 tot 5 m/d). If their permeability is low, they have an open drainage system of furrows; the other soils can be subsurface-drained.

The first-mentioned fluviatile soils are used as arable land, orchards and grassland; the other soils are used as grassland and locally some willow coppice.

3. *The low-land peat soils*

The peat varies greatly depending on its origin. The Holocene peat lies on a Pleistocene sand layer and varies in thickness from < 1 m (especially in the north) to > 10 m (especially in the west). The peat layer is often covered by a thin clay layer, which largely determines the suitability of the soil. The permeability factor of the peat ranges from 0,05 to 2,50 m/d. These soils are used almost exclusively as grassland.

4. *Sandy soils*

The sandy soils mainly consist of fine sand (diameter of the sand grains 10 to 210 µm). The percentage of silt (2 to 50 µm) is predominantly lower than 20 per cent and the percentage of organic matter in the plough layer usuall varies from 3 to 7 per cent. The permeability factor of these soils usually ranges from 0,05 to 2,50 m/d and decreases as the silt content increases.

In the north of the country, glacial till is often found at a shallow depth this poorly permeable, medium-textured material determines drain space. Sandy soils are suitable for both arable land and grassland, but are mainly used as grassland.

2.4 Land use

The Netherlands, with a population density of more than 400 inhabitants per square kilometre, is one of the most densely populated countries in the worl The demand for land, for various purposes, is therefore great. Apart from th need for agricultural land, a great deal is also needed for town planning, road building, establishment of industry, nature conservation, open countrysides and outdoor recreation.

This great demand has led to high land prices and to the necessity of making the available land area as suitable as possible for the purposes concerned. The Government plays an important role here both in town and country plannin as well as in land development.

The shortage of agricultural land has led to an intensive use of the soil. Table 4 gives some data to illustrate this.

TABLE 4. Some data on the intensive use of soil

Description	Data
Price of average good freehold farmland	ƒ 2,50 to ƒ 5,00 per m^2
Farm animals	2 to 3 per ha
Average yield	
a) potatoes	40 ton per ha
b) sugarbeets	55 ton per ha

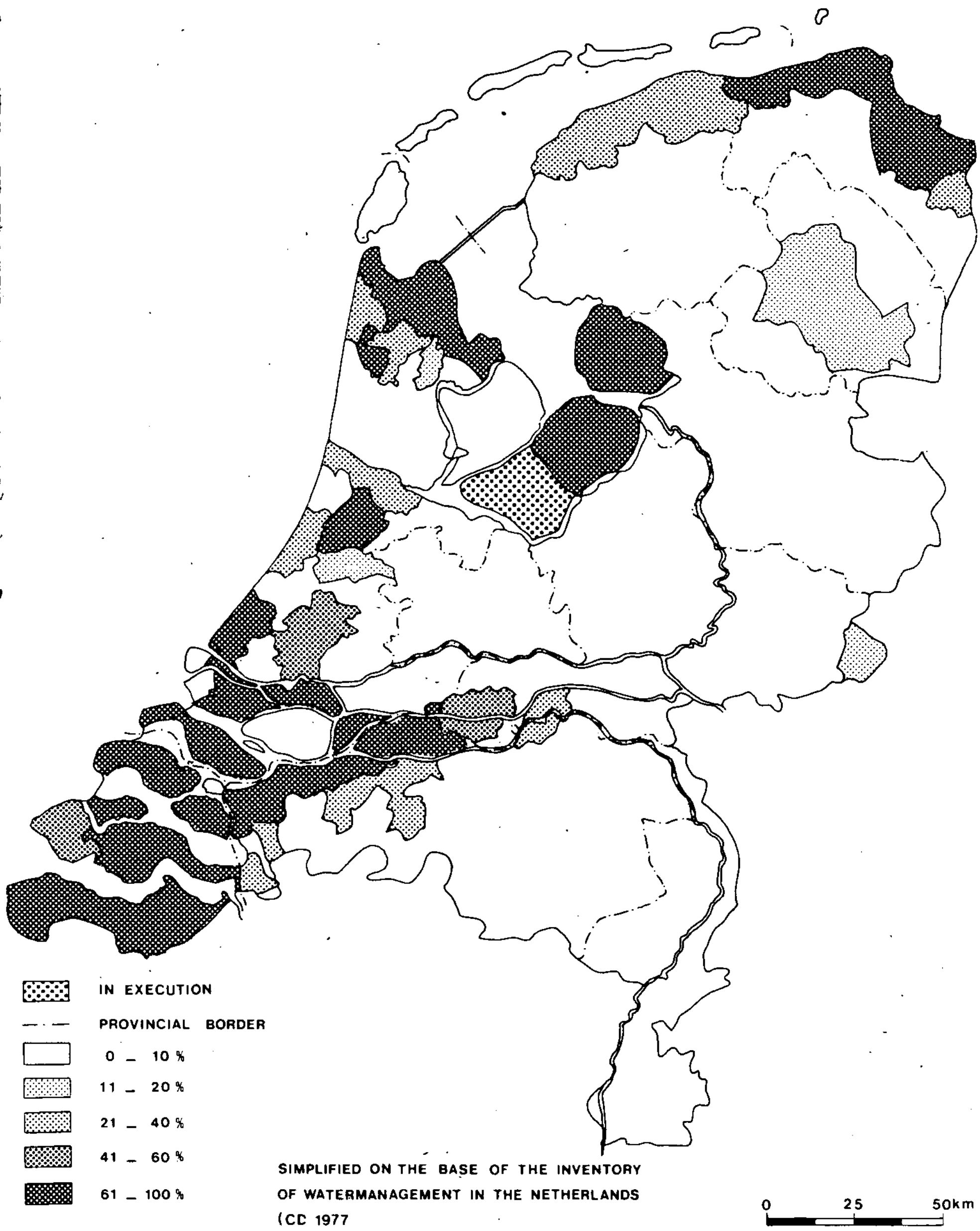

Fig.3. Proportional extent of subsurface-drainage in The Netherlands.

In order to make this possible, optimum planning of the farmland is essential. Together with other land development schemes, such as plot enlargemen the introduction or renewal of subsurface drainage is also important.

3. Present state of affairs

3.1 Drainage requirements

For many years The Netherlands were drained by ditches. Since the middle of the last century, subsurface-drainage has gradually become more important. The poor control of the water levels in the ditches and watercourses, into which the subsurface drains flowed, was a particularly serious problem. The improvement in this level control and the installation of subsurface drainage must be coordinated. The extent of subsurface-drainage in agricultural land, according to the situation in 1977, is shown roughly on the map reproduced on Fig.3. This map shows that most of the drainage is found in the sea-clay soils of the north and south-west of The Netherlands. The remainin more widely dispersed, intensively drained areas are situated in reclaimed lakes and land consolidation schemes already completed. An accurate prognosis of the area requiring drainage is difficult. It depends on the following factors:

a) the costs/benefit ratio for the intallation or renewal of subsurface drainage;

b) the degree to which the installation of subsurface-drainage become possible through other measures, such as lowering the water levels in the ditches and enlarging the plots by filling in the ditches.

Roughly estimated, subsurface-drainage must be introduced or replaced in another 5 000 km^2 *(500 000 ha)*. If the annual area of subsurface-drainage of about 25 000 ha remains unaltered, then, on the basis of this prognosis, sub surface-drainage must continue to be installed for another 20 years. Fig.4 gives a general impression of the situations where drainage pipes still hav to be introduced or renewed.

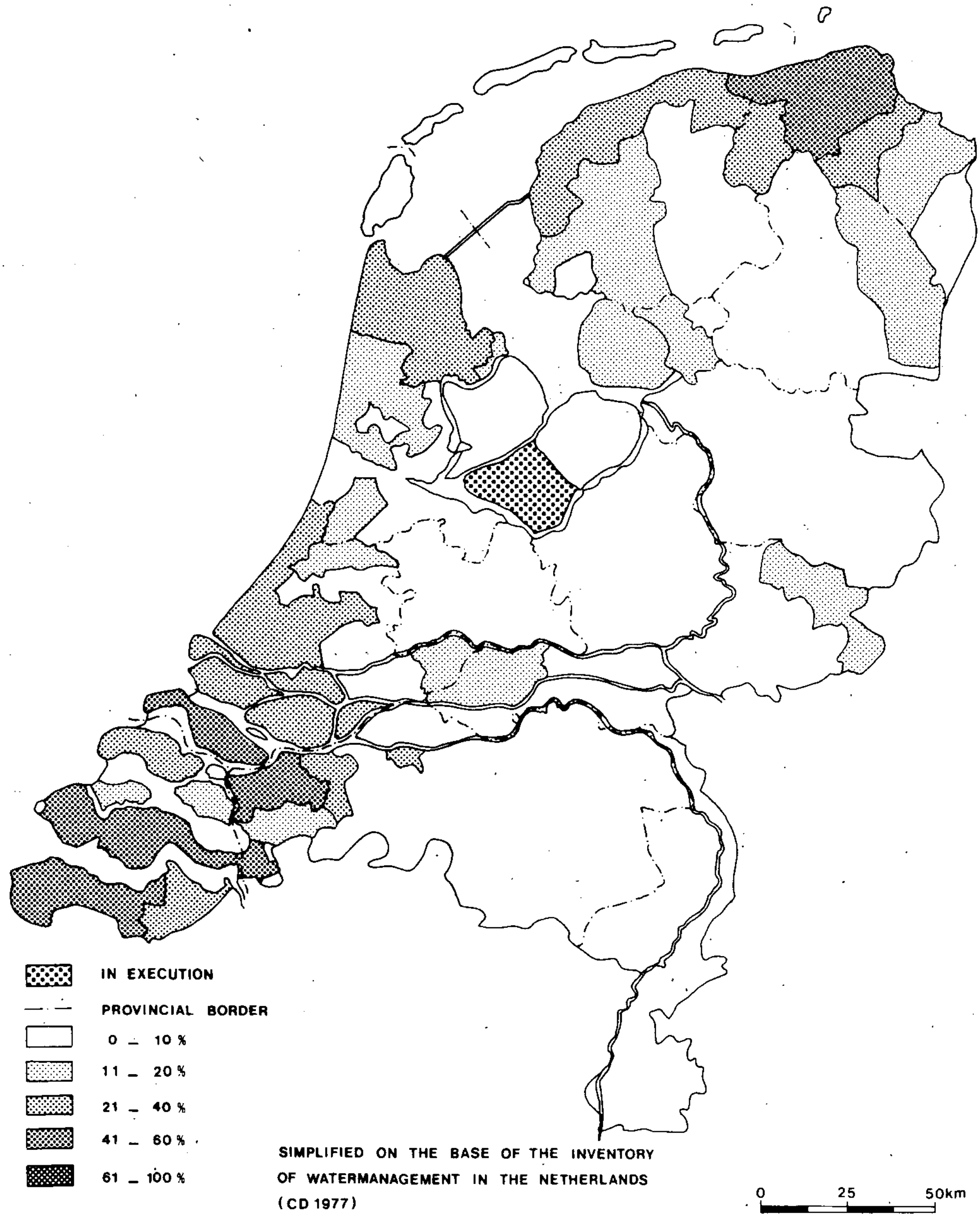

Fig.4. Proportional extent of arable land in The Netherlands where subsurface-drainage has to be constructed or renewed.

When both maps are compared it can be seen that the most intensive subsurfac drainage in the future will have to be laid in those areas where there is al ready a great deal of subsurface-drainage. Replacement of existing subsurfac drainage is essential. The introduction of new subsurface-drainage must be expected particularly in the north and west of the country. These are mainly the upland reclaimed peat areas (Veenkoloniën) and the low-land peat soils with or without a clay cover (0.1 to > 1 m). The installation of new subsurface-drainage in the river-clay areas and in sandy soils is expected to a lesser extent.

It is essential to note that both maps are based on average data for large areas. This results in strong deviations in detail. The large areas where, according to the prognosis, less than 10 per cent of the area requires subsurface-drainage, must not be ignored when determining the total area for subsurface-drainage.

3.2 System and criteria

In The Netherlands, the singular system is used almost exclusively. The laterals, the lengths of which are usually 150 to 250 m, flow by means of outlet into ditches or watercourses. The depth at which the drains are laid vary from 1,00 m to 1,30 m, while the slope is usually 1:1000.

The design criteria for agricultural subsurface-drainage which have always been handled by the Government Service for Land and Water Use are formulated as follows. With continuous drainage of 7 mm/d, the difference in height between the groundwater level and the ground level may not be less than:

a) for arable land 0,50 m
b) for grassland 0,30 m

The norm used for the Lake IJssel Polders is a height difference of 0,30 m with a drainage of 10 mm per day. The distances between the drains, determined on the basis of the criteria vary in general:

for sea-clay soils	from 10 to 20 m
for river-clay soils	from 10 to 20 m
for low-land peat soils	from 10 to 30 m
for sandy soils	from 15 to 30 m

3.3 Materials

The pipes

A summary of the various pipes used in The Netherlands since 1958 is given in Fig.5. Until about 1960 only clay tiles were used. The specifications for these pipes are described in NEN 440. The usual internal diameter was 50 mm.

Between 1960 and 1970, slotted smooth PVC pipes were used, usually with external diameters of 40 mm and 50 mm (NEN 7010). The most important advantage of this pipe, compared with the clay tile,was that the transport and installation costs were much lower. In about 1968, corrugated perforated PVC drainage pipes appeared. This material took over practically the whole market within five years. The reasons being that:

- corrugated pipes performed equally well as clay tiles
- the material was cheap, labour saving and allowed a greater driving speed for mechanized draining.

Cover and envelope materials

Cover and envelope materials are usually applied for two reasons:

- to stimulate the flow of groundwater to the pipe
- to prevent infiltration of residual soil particles into the pipes.

Loose organic materials, mainly peat-fibre and to a lesser extent heather and straw were used in the past. After a transitional period, during which strips of organic materials were used, the complete enveloping of the pipes is now practised almost exclusively.

Table 5 gives the average use of cover and envelope materials in land consolidations from 1973 to 1977. The use of material outside the land consolidations is comparable.

ANNUAL FIGURES FOR THE USE OF VARIOUS TYPES OF DRAINAGEPIPES IN THE NETHERLANDS

FIG. 5

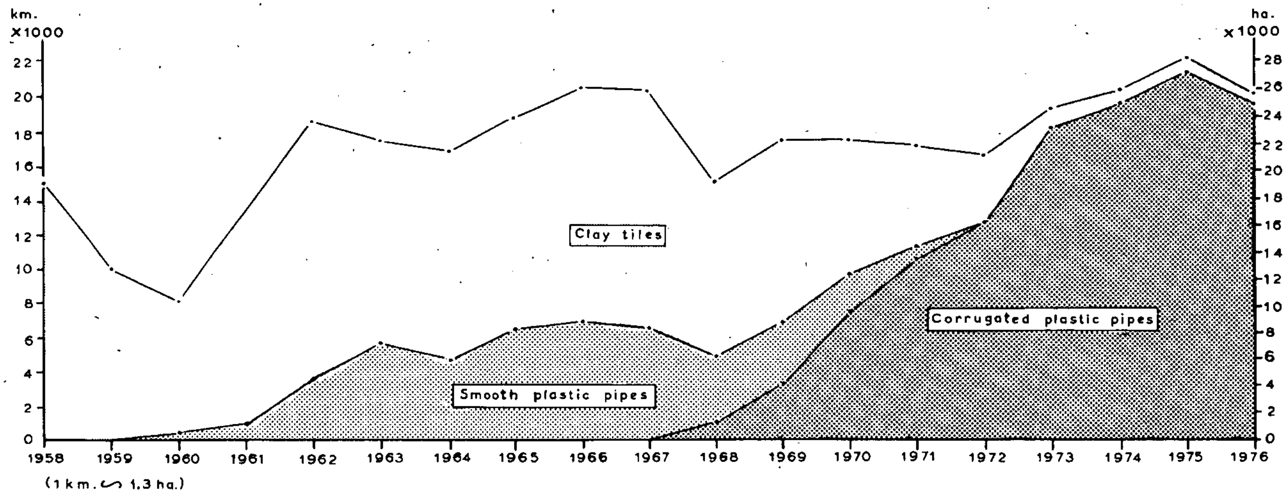

TABLE 5. Application of drain cover and envelope materials in land consolidations from 1973 to 1977

Cover and envelope materials	Percentage of the length
peat-fibre	9
flax	3
coconut-fibre	73
fibreglass	10
other	<1
without	5
Total	100

This table shows that only 5 per cent of the length of subsurface-drainage has no form of cover or envelope material, while 85 per cent is enveloped in voluminous organic materials. Coconut-fibre, with 73 per cent, is the main material (Photo 2). The thin, synthetic glassfibre, used to a limited extent in the south-west, is wrapped in a simple way round the pipe during drainage. The other materials such as coconut-fibre (NEN 7047), peat-fibre (NEN 7048) or a mixture of both are wrapped round the pipe in the factory.

This has the disadvantage of hampering inspection of the pipe during installation. There are certain drawbacks to the existing variety of materials, the most important being:

a) a limited variety is rather vulnerable as far as supplies are concerned. Good peat-fibre is scarce;
b) organic materials decompose which in certain cases can be detrimental to satisfactory functioning;
c) organic materials often differ in composition and quality;
d) the thin fibreglass has limited application.

In the last five years, therefore, there has been an intensive search for new materials, resulting in 1977 in some new voluminous synthetic cover and wrapping materials being accepted. One of which is a layer of polypropylene fibres wrapped round the pipes in the same way as the coconut-fibres and the other a layer of polystyrene granules wrapped round the pipes and kept in place with perforated plastic foil (Photo 3).

3.4 Machines

Subsurface-drainage in The Netherlands was, until about 1955, installed exclusively by hand. Since then and within only a few years, machines have completely taken over and many technical improvements have since been made to these specific draining machines, greatly improving their productivity. This development is illustrated in Table 6.

TABLE 6. Developments in draining machines

Features	1955	1977
engine power per machine	ca 35 kW (ca 45 hp)	ca 150 kW (ca 200 hp)
number of laboures per machine	ca 11	3 or 4
capacity per week per machine	5 to 7½ km	15 tot 25 km

With these developments the technical possibilities of draining machines available in The Netherlands greatly depends on the year they were built (Photo 4 and 5). A few features of the modern trenchers are:

a) complete traction on tracks (hydrostatically driven);
b) engine power 125 to 175 kW (170 to 235 hp);
c) excavator chain (usually semi-vertical, mechanically driven);
d) working depth of excavator chain for Dutch conditions to about 1,80 m;
e) hydrostatic adjustment of the excavation depth;
f) depth regulation by means of a rotating laser beam.

Only a few drains have been laid by trenchless machines (Photo 6) because studies have shown that such drains - with the exception of those in sandy soils - are not as effective as those laid by trenchers. These remarks do not apply to the Willner trenchless machine which is a favourable exception (Photo 7). A rough estimate shows that about 75 trenchers are used by about 40 specialized Dutch drainage-firms.

3.5 Costs

An estimate of the costs of subsurface-drainage is given in Tables 7 and 8.

Table 7. Costs of subsurface-drainage in ƒ per m in 1977

Material			Work	ca 10% of 3 and 4	Total
pipes	envelope	cost			
1	2	3	4	5	6 = 4 + 5
diameter 60 mm	none	0,70	0,60	0,15	1,45
	coconut-fibre	1,15	0,60	0,20	1,95
	peat-fibre	1,40	0,60	0,20	2,20
	glassfibre	0,90	0,60	0,15	1,65
diameter 80 mm	none	1,25	0,60	0,20	2,05
	coconut-fibre	1,95	0,60	0,25	2,80
	peat-fibre	2,45	0,60	0,30	3,35
	glassfibre	1,50	0,60	0,20	2,30

Included in the costs are:

delivery of the material;

execution of the work without any help from the client;

overheads such as the contractor's costs for administrative and executive personnel, profit and risks; these costs are calculated as a percentage of the costs of materials and labour.

Not included are the costs for:

fittings such as outlets (ca ƒ 0,05/m drain);

costs of drawing-up the plan and supervision costs (varying from 0 to 20 per cent);

value-added tax (usually 18 per cent, sometimes partially 4 per cent).

Table 8 gives an approximation of costs for the much-used 60 mm corrugated PVC pipes.

TABLE 8. Costs of subsurface-drainage in *f* per ha in 1977, with a drain diameter of 60 mm

Envelope material	Costs per m	Distance between drains in m		
		10	20	30
		length of drains in m per ha		
		900	450	300
1	2	3	4	5
none	1,85	1 665	835	555
coconut-fibre	2,45	2 205	1 105	735
turf-fibre	2,75	2 475	1 240	825
glassfibre	2,10	1 890	945	630

The costs in Table 8 are based on the prices given in Table 7, increased by *f* 0,05 per m' for fittings, by 10% for the costs of drawing-up the plans and for supervision and by an average of 12% for value-added tax.

4. Agricultural applications

4.1 Farming

Most of the subsurface-drainage work in The Netherlands is commissioned by either owners or users of agricultural land. The scale of operations is usually small; most of the orders are for 3 to 10 ha. The agreement procedure between the client and the drainage contractor is straightforward and usually follows this pattern:

the drainage contractor estimates the costs;

when the contractor and the client have reached an agreement, it is confirmed in writing;

after the work has been completed the agreed amount is paid.

A variation on this procedure and one which occurs rather often, is when an engineering bureau, specializing in land development plays an intermediary role. Since 1975 a Government policy has existed to help with the costs of private land development; this also includes subsurface-drainage. The

Government subsidies amount to 30 per cent of the costs and are granted by the Government Service for Land and Water Use.

4.2 Land development

Land development in The Netherlands means either developing or improving rural areas for new or existing land uses. At present there are several forms of land development either in preparation or actually in operation. The most important operational form is that of land consolidation. The aims of land consolidations vary from area to area, but are mainly the following:

a) to increase the accessibility of rural areas by building or improving country roads;

b) to improve water management by introducing or improving watercourses, weirs, pumping plants, etc.;

c) to improve parcelling by re-allocating plots and shifting farm buildings;

d) to protect and restore the countryside by planting and conserving trees and shrubs;

e) nature conservation by, for example, transferring land ownership to the Government or to a nature conservation society;

f) to provide facilities for outdoor recreation.

A land consolidation project is prepared by representatives of the area concerned in conjunction with the Government Service for Land and Water Use. The owners and tenants decide on a land consolidation scheme by vote.

The work for a land development scheme is done by contractors and the project is directed by land development engineering bureaus; the Government Service for Land and Water Use has the general supervision and pays the costs.

The average costs of a land consolidation scheme are about ƒ 5 000 to ƒ 6 000 per ha. About two-thirds of these costs are covered by the Government. The rest is temporarily financed by the Government but must be paid back by the owners within 26 years after the transaction.

Together with these improvements in water management and parcellation, superfluous ditches can be filled up and other ditches and watercourses are either enlarged or introduced. Subsurface drainage is also often introduced at the same time. A specific problem, however, is that the subsurface-drainage works and land division works can hamper each other. It is then inevitable that the old drainage system is eliminated by filling in the ditches before the new subsurface-drainage is installed. In the intervening period, however, there is no drainage system at all and with heavy rain serious damage and delay could occur.

This risk is kept to a minimum by:

a) working as much as possible in summer;

b) giving all the responsibility to one contractor, thus ensuring the smooth running of the work.

One disadvantage is that a great deal of compensation must be given to the farmers if they are not able to use their land during the growing season.

4.3 Land reclamation

Several polders are being formed in Lake IJssel which, after draining, are parcelled by the IJsselmeerpolders Development Authority. In this land division the following points regarding groundwater control are considered.

1. The number of ditches is kept to a minimum.
2. The subsurface-drainage system is singular.

In the last few years, this has led to plots with a length of about 1300 m and a width of about 500 m; the length of the subsurface-drains being about 250 m. One special aspect of draining in the Lake IJssel Polders is that the soils mature after they have been drained. In the course of maturing the drained soil cracks by shrinkage above the groundwater level. Because these cracks are permanent, they improve permeability. This permeability largely depends on the texture of the soil. The plan for subsurface drainage is drawn-up in accordance with a standardization of the soil profile and using results obtained from experimental plots with varying drain distances and drainage materials.

5. Non-agricultural applications

Non-agricultural-applications will be discussed only briefly. This will be done in such a way that the more interested reader can find some guide for closer orientation.

5.1 Outdoor recreation

When planning those areas that are going to be used for outdoor recreation in one form or other, the lowering of the groundwater level is often just as important as in agricultural areas.

Design standards used in the application of subsurface-drainage are given in Table 9.

TABLE 9. Drainage criteria for outdoor recreation sites

Purpose	Amount of water which has to be continually drained (mm/d)	Permissible minimum drainage depth (height difference between ground level and ground-water level) (m)
Playing-fields	15	0,5
Lawns for ballgames	15	0,5
Camping sites	10	0,5
Pleasure parks and playgrounds	7	0,3 to 0,4

Besides, measures must be taken to prevent water stagnation on the field or in the top layer. There are no general standards for this in The Netherlands. As the use becomes more intensive, more measures will be adopted.

Humus-deficient and silt-deficient sand in thicknesses of 0,05 to 0,20 m is deposited over the toplayer to store the water temporarily during and directly after heavy rain.

Where the permeability of the subsoil is poor and cannot be improved the sandy top layer is brought into contact with the subsurface-drainage by filling the drainage trenches with easily permeable material (usually coarse sand). This ensures that water, which could stagnate on the poorly permeable subsoil, can flow to the drainage trench via the top layer (Photo 8).

In general there are no important differences in material and work compared to agricultural applications of subsurface-drainage.

5.2 Construction of roads

Subsurface-drainage is applied under three different circumstances in road building. These are: lowering the groundwater level; preventing perched groundwater tables; draining surface water.

a) Lowering the groundwater level

Sand is usually used for improving the subsoil and for raising the road-bed. In order to obtain a subsoil with a satisfactory bearing capacity, the groundwater level must be sufficiently deep beneath the hardsurface. For very intensively used roads this is at least 0,80 m. Ditches or subsurface-drains are used here.

b) Preventing perched groundwater tables

The most frequently occurring form is the stagnation of water on the bottom of the sand-bed. By installing, along the road, one or more subsurface drains into this sand-bed, these perched groundwater tables can be prevented quite easily (Fig.6).

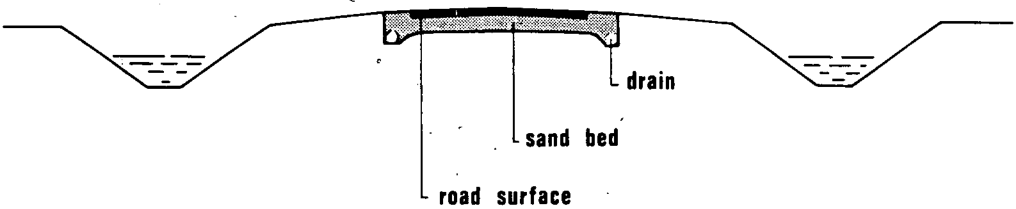

Fig.6. Drainage of sand-bed.

c) Draining surface water

The drainage of surface water is not discussed because subsurface drainage for this reason is only applied in The Netherlands in exceptional cases.

5.3 Town planning

New urban and industrial areas are often built in places with groundwater levels that are too high and with soils that have insufficient bearing capacity.

From 1950 to 1970, such sites were usually made more suitable by applying 2 to 5 m sand. This sand is usually taken from the subsoil elsewhere and to the site through pipelines by sand pump dredgers. Because this method was very expensive, another solution was needed. This was found by lowering the groundwater level by subsurface drainage. Raising with sand is nowadays only necessary to achieve a subsoil with a high enough bearing capacity, in order to make construction work possible. The required thickness of the layer can now be reduced to about 1 m. A serious problem here is however that construction work, for example pile-driving for foundations and sewage work, disturbs the subsurface drainage. Additional subsurface drainage installed after completion of the work is expensive.

The IJsselmeerpolders Development Authority has developed a solution that has recently been used in the area being prepared for the construction of the new town of Almere in the Lake Yssel Polders. It consists of an interlocking drainage system. After the first drainage system has been laid at a depth of 2 to 2,5 m below the future ground level, a second system, perpendicular to the first, is laid about 0,2 m higher. Because a layer of well permeable material (lavaliet) is put about the pipes in both systems, they are linked to each other at the points of intersection. The fact that some disturbances occur with this system is accepted, but with the intersecting network, drainage is almost always possible (Fig.7).

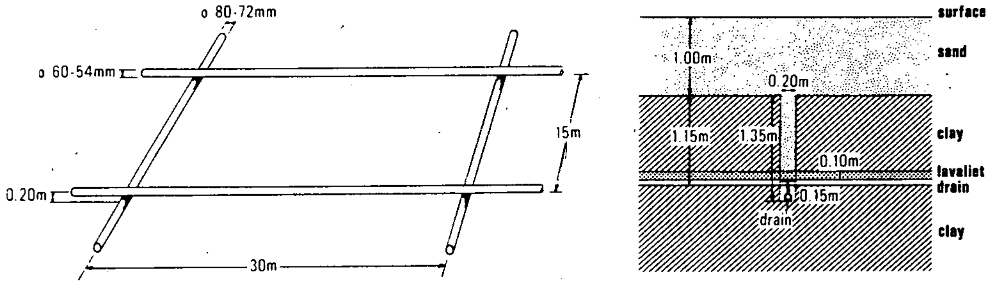

Fig.7. Interlocking drainage system in Almere.

Deep drainage should be mentioned as an exceptional form of subsurface drainage. This technique, in which the drains can be accurately laid at a depth of 4 to 4,5 m, uses chain trenchers fitted with a shaft behind the excavator chain. By means of this shaft the flexible pipe can be directed to the bottom of the trench. This technique is used for example in draining the sand-raised sites.

Another application is the temporary lowering of groundwater levels, especially when laying underground pipelines for the transport of liquids and gasses. Subsurface drainage, which will be linked up with a pumping installation, is first placed below the level of the pipelines. Since accuracy in laying is less important here, depths of 5 to 7 m can be reached.

5.4 Vertical drainage

Vertical drainage is used in the west of The Netherlands for urban development schemes and construction of roads, when situated on strongly compressible subsoils such as an clay-peat layers. This method of drainage speeds up the settling process and therefore ensures that settling after building remains limited.

If a strongly compressible subsoil is overloaded, for example with a sand layer for a building ground, then settling will occur. Settling means that the volume of pores must decrease. If the pores are saturated with water, it hampers the volume-decrease of the pores. It is only if the water can flow away that settling occurs. Because the pores are very small (permeability is low) and the relative distance that the water in the pores must cover is great, drainage and settling will be very slow.

This flow can be improved by connecting the water in the layer vertically with the surface and with the easily permeable sandy subsoil. By using a drill-jet, these connections can be achieved by drilling a hole in the layer and then filling it with easily permeable sand. These sand-drains, with a diameter of 0,2 to 0,3 m are laid 2,5 to 3,5 m apart in a pattern of equilateral triangles.

For a few years, synthetic materials have also been used for this vertical drainage in The Netherlands. A strip of synthetic material is placed in the layer and the water forces its way into the strip and flows up or down the length of the strip.

6. Institutions involved in the research

The following sections summarize the most important institutions involved in drainage research and their activities.

Ministry of Agriculture and Fisheries

a) The Government Service for Land and Water Use in Utrecht is involved in subsurface drainage, through its connection with the preparation and supervision of land development projects. Its work on subsurface drainage is mainly in planning and judging plans made by others. Scientific research is also supported by field trials.

b) The Institute for Land and Water Management Research in Wageningen, which is closely associated with the Government Service for Land and Water Use, studies, land development, water management, water quality and soil improvement.

c) The International Institute for Land Reclamation and Improvement in Wageningen is active in scientific research, education and research projects for developing countries. Subsurface drainage is one of the subjects dealt with.

The mutual interests in subsurface drainage of the institutions mentioned, are discussed in the *Drainage Contact Group*.

d) The Agricultural University of Wageningen. Drainage is part of the curriculum and research programme of the Department of Land and Water Management.

Ministry of Transport and Public Works

The IJsselmeerpolders Development Authority in Lelystad is responsible for the preparation and planning of the Lake IJssel Polders and a few other new areas.

This body is active in scientific research and in the planning and execution of projects including subsurface drainage. The Authority has experience both in agricultural and urban drainage.

Trade and Industry

a) For general research into subsurface drainage, the major engineering bureaus dealing with land development are the following:

Grontmij in De Bilt
Heidemij in Arnhem

Their main work is preparation of projects and research necessary for the execution of the projects and supervision of the work done by contractors.

All these institutions maintain informal contact in the Drainage Study Group.

b) Trade and Industry are active in research and development of draining machines and drainage materials. This research is usually done by private industry.

c) The KOMO Institute in Rijswijk is an establishment for research, standardization and inspection of materials and constructions. This Institute is involved with the following activities concerning subsurface drainage.

I. Deciding the specifications for drainage materials. This is done in consultation with trade and industry and various Government services. These specifications are published in the form of Dutch Standards papers (NEN) by the Dutch Standardization Institute. They are included after international consultation on specifications.

II. The granting of a quality guarantee mark to factories, whose production processes are continuous tested by the KOMO Institute.

III. The inspection of materials.

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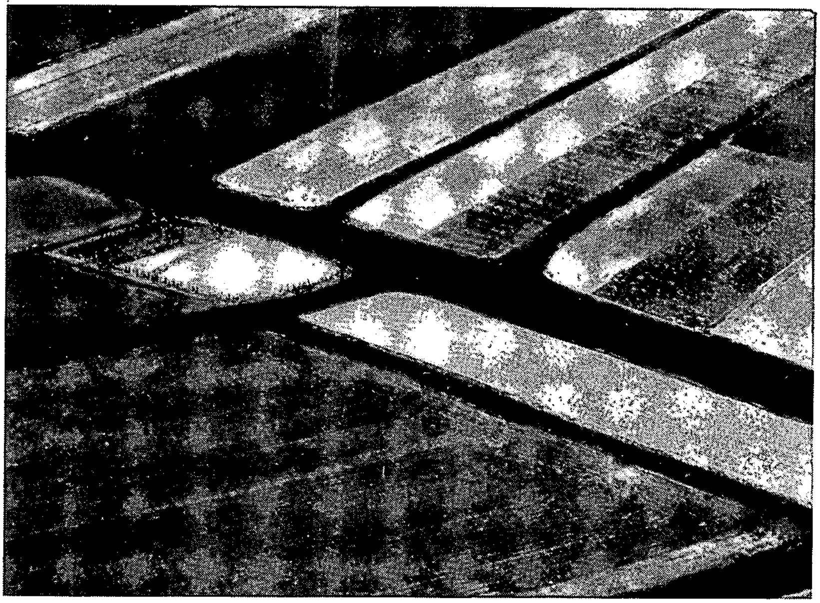

Photo 1. An area rich in ditches.

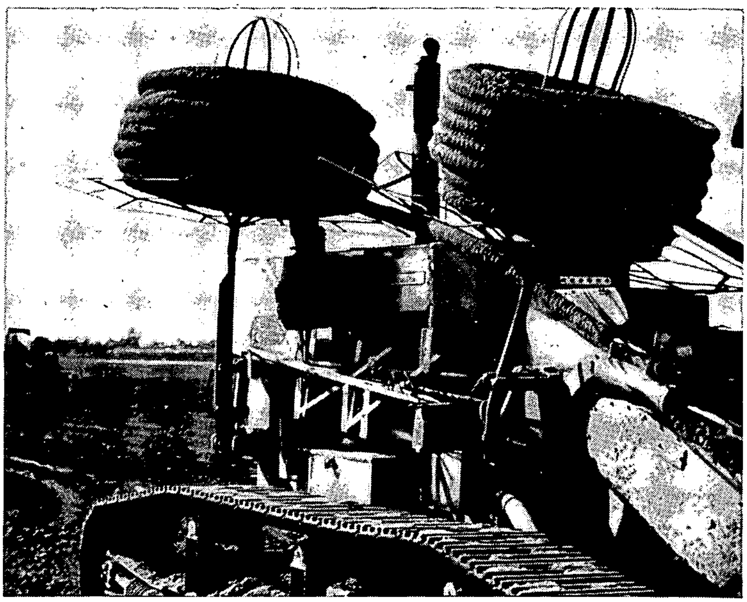

Photo 2. Rolls of corrugated PVC pipes enveloped in coconut-fibre.

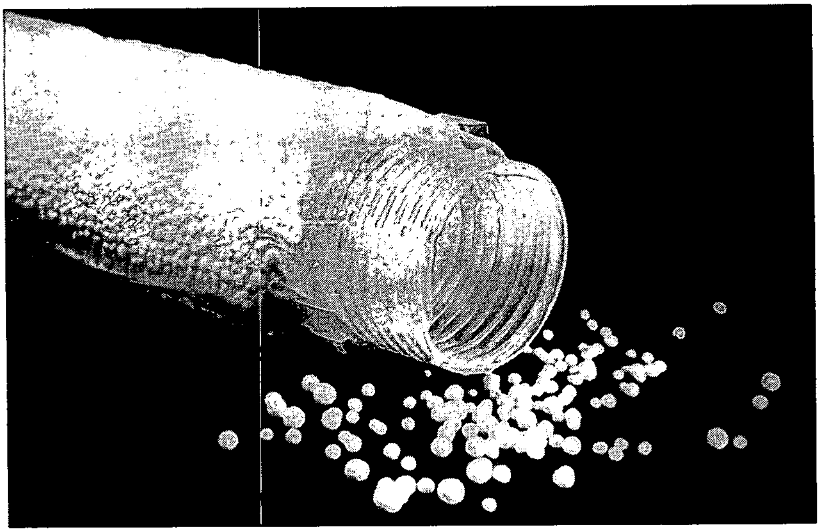

Photo 3. Polystyrene granules as envelope material.

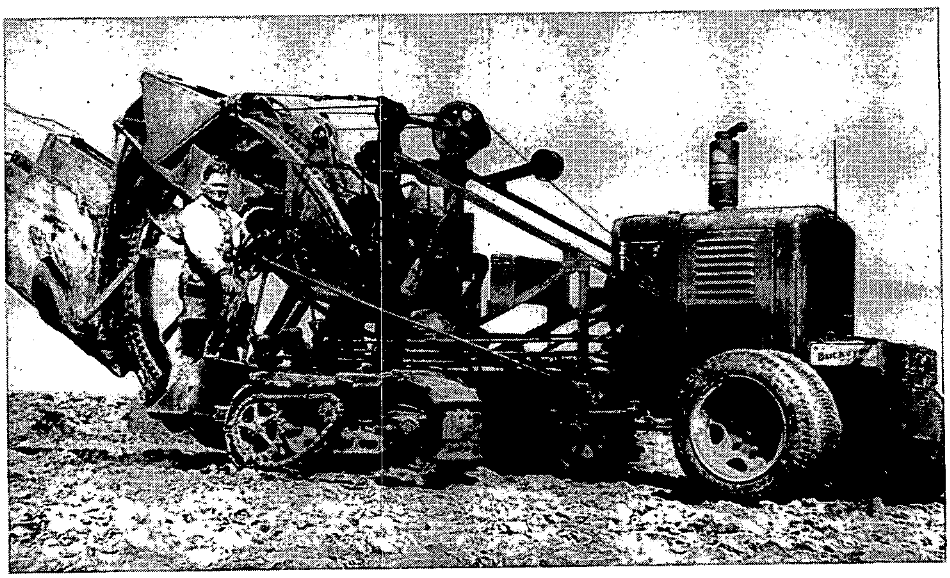

Photo 4. Trencher (model 1955).

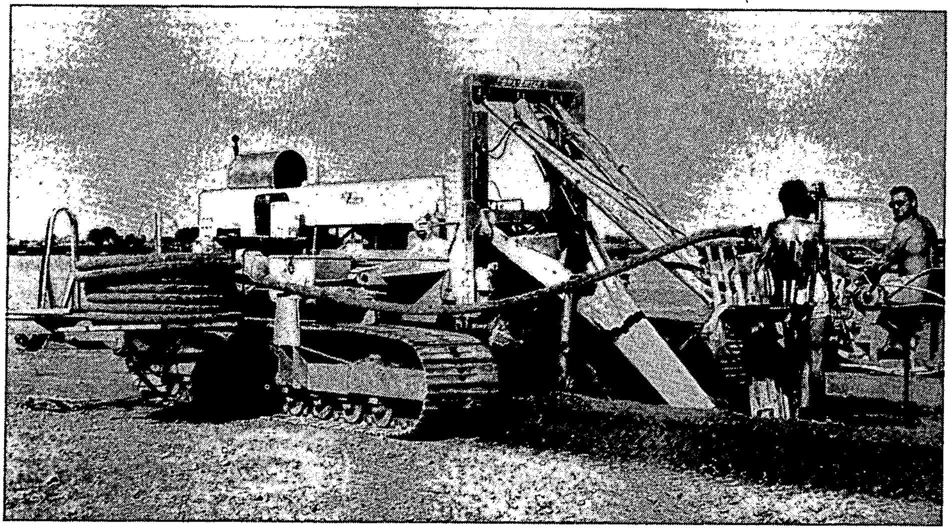

Photo 5. Trencher (model 1975).

Photo 6. Trenchless machine.

Photo 7. Willner trenchless machine behind caterpillar tractor.

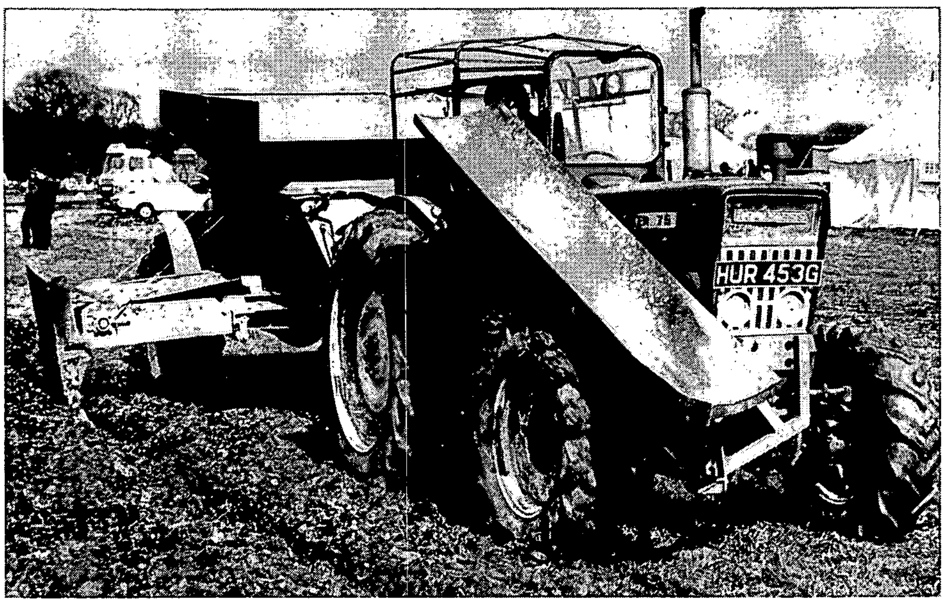

Photo 8. Ditch-filling with easily permeable material.

Paper 3.04

DEVELOPMENTS IN LAND DRAINAGE

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Summary

The change from clay to smooth plastic and then to corrugated P.V.C. pipes greatly reduced the labour required and the cost of transport on the site, and advanced mechanization, technical developments in cover and envelope materials also did much to further mechanization and rationalize drainage. As a result of these factors, drainage has been mechanized to such a degree that it requires a minimum labour force.
Developments in the field of automatic depth regulation systems have also contributed to this process.
Because of the increased mechanization and rationalization of agriculture, the need for drainage has risen sharply, but drainage maintenance in some places is far from satisfactory; it is most essential that instruction should be provided with regard to maintenance.

1. Introduction

In the course of water control board, land consolidation and reclamation works, land drainage has been - and is being - improved in large areas of The Netherlands. As a result of improved land drainage, land consolidation and reclamation, large areas of agricultural land are being drained. Since 1967 about 20,000 hectares (200 km^2) per year have been provided with a land drainage system. In 1975 interest was aroused in the private sector by the possibility of grants for private civil engineering projects. The laying of pipes, like other civil engineering activities, has been transformed in the past 20 years from a manual to a fully mechanized operation.

The range of materials has undergone a similarly swift transformation. Before 1956 practically all pipes were made of vitrified clay. In that year the first tests were carried out with plastic pipes.

The replacement since 1961 of loose peat fibre by new cover and envelope materials has also brought about vast changes. This ready acceptance of new possibilities obviously means that practical application often outstrips research. This, however, may easily produce disappointing results, and indeed has done so in some cases. Laboratory research must concentrate in the future on making a sound pre-selection of those materials that are suitable for practical use. When drainage networks were installed in areas that used to be drained piecemeal by ditches and drainage ditches, sufficient attention was not always paid to providing instruction about drainage maintenance. Consequently the functioning of the drains in these areas sometimes gave rise to dissatisfaction.

2. Drainage pipes

Up to 1956, as we have said, practically all the pipes used were collarless and collared clay pipes and, these were in short supply, concrete pipes. In collaboration with the Heidemaatschappij, field research was carried out in that year with smooth plastic pipes. Of this development Van Someren wrote (Med. No.4 CD, 1959):

> *"It looks as though smooth plastic pipes of the specifications and perforation now in use do not, to say the least, function less well than the clay pipes. Their drainage qualities are not the only important factor, however. As regards installation costs, too, the smooth plastic pipe opens up new prospects for the near future, because it increases the possibility of further mechanization and thus a saving of labour costs. This particular aspect will gain progressively in importance, so that clay pipes, which have had a monopoly for about a hundred years, will soon find it very difficult to compete with plastic pipes".*

In 1959 the first smooth plastic pipes were put into practical use; in 1962 they were already used for 20% of the area drained that year, and for 33% between 1963 and 1965. The subsequent development is shown in Table 1, which gives the pipe materials used in land consolidation projects.

TABLE 1. Length of drainage in land consolidation projects

Year	Pipe material							
	corrugated plastic		smooth plastic		clay		total	
	km	%	km	%	km	%	km	%
1966			1474	52	1354	48	2828	100
1967			1515	52	1400	48	2915	100
1968			1843	60	1252	40	3095	100
1969	208	5	2192	56	1502	39	3902	100
1970	422	11	1728	42	1894	47	4044	100
1971	1649	43	1041	27	1157	30	3847	100
1972	1550	37	679	16	1939	47	4168	100
1973	2879	93	-	-	216	7	3095	100
1974	2111	100	-	-	-	-	2111	100
1975	3648	100	-	-	-	-	3648	100

After about 1967, when drain covering strips were developed and put to practical use, it was no longer possible to check whether the joints of smooth plastic pipes had come adrift. The firm of Wavin therefore produced a pipe with a fitting at the ends which made the joints secure. This proved satisfactory provided the machine was manned by a reliable pipe layer.

In 1963 a completely new type of pipe started to be manufactured: corrugated PVC pipe. It had the same compressive strength as the smooth plastic pipes, but because of the corrugation took much less raw material. Another important advantage was - and still is - its great flexibility, which enabled it to be delivered and used in rolls of a manageable diameter and considerable length. This also had a beneficial effect on the amount of labour required.

The first field tests with these pipes were carried out round about 1964. In 1969 the production of corrugated pipes got well under way, and they were used for 5% of land consolidation drains. By 1971 this proportion had risen to 43%, and at present practically no other pipes are used. The transition from clay to smooth plastic and subsequently to corrugated PVC pipes has considerably reduced transport and labour costs, besides promoting mechanization.

3. Cover and envelope materials

When drains are laid, a highly permeable material is often put on or around the pipes. Its function is discussed in another paper (2.15) entitled "Guidelines for the selection of envelope materials for subsurface drainage".

In former times the cover material most commonly used was peat fibre. The job of delivering it, transporting it along the drain trenches and putting it on the pipes was time-consuming and labour-intensive. When smooth plastic pipes were introduced, the job of transporting them on the site was simplified because they were lighter and more of them could be transported per trip. Investigations were consequently started to find out if economies could also be made in the transportation and laying of cover material, which would help to rationalize drainage even more.

At first glass fibre was thought to be suitable for the purpose: the pipes could either be delivered prewrapped, or it could be wrapped round them on the drainage machine. At that time clay pipes were already wrapped in glass fibre, one strip being placed underneath and one on top. Field tests soon showed, however, that glass fibre had certain drawbacks in fine sandy soils. In these soils it was vital that the permeation surface should be increased and this could only be done by using a thick layer of cover of envelope material. A means therefore had to be found of producing voluminous materials in strip form, in order to achieve the same efficiency on the site. The firm of Ripken and the Griendtsveen Turfstrooisel Maatschappij took the initiative in manufacturing peat fibre strip. To enable this to be used on the drainage machine, the firm of Van der Ende of Tiel designed a reel which could hold four rolls of peat fibre.

Another completely different voluminous material that was applied in loose form was flax fibre, a waste product from flax industry. From 1962 onwards, bales of this material were marketed by Horman of 's-Gravendeel. It was used mainly in the sea clay region of North Brabant. By 1968 the product had been sufficiently developed to be delivered also in strip form.

In 1972 coconut fibre strip was introduced. It very soon became widely used, and by 1975 coconut fibre strip and pipes prewrapped in it had practically captured the market. This can be seen from Table 2, which gives the length of drains laid in land consolidation projects and the cover and envelope materials used in percentages of the total length.

TABLE 2. Length of drains laid in land consolidation projects and the cover and envelope materials used in percentages of the total length

Covered and envelope material	Pipe material														
	corrugated plastic						smooth plastic						clay		
	covered %			wrapped %			covered %			wrapped %			covered %		
	69	72	75	69	72	75	69	72	75	69	72	75	69	72	75
loose peat fibre	–	5	–	–	–	–	21	2	–	–	–	–	15	19	–
peat fibre strip	3	8	–	–	–	8	21	–	–	–	–	–	7	–	–
flax fibre strip	–	12	<1	–	–	–	–	1	–	–	–	–	–	17	–
coconut fibre	–	–	15	–	–	84	–	–	–	–	–	–	–	–	–
glass fibre	–	8	–	2	–	7	14	12	–	–	–	–	6	0	–
other	–	0	–	–	–	<1	–	–	–	–	–	–	11	–	–
none	–	4	–	–	–	–	–	–	–	–	–	–	–	11	–

This swift rise in the use of coconut fibre led manufacturers, for competitive reasons, to market it in different thicknesses, which was not beneficial to the quality of drainage work. In order to avert the dangers involved the Government Service for Land and Water Use took the initiative in establishing quality requirements for cocnut, flax and peat fibre cover and envelope materials. These requirements, which were applied for some years on all the Service's civil engineering projects, were adopted by the Filter Material Committee, under the auspices of the KOMO, as the official requirements for all public works involving drainage. They apply to peat, peat/coconut and coconut fibre, but not to flax because it has been used so little in recent years. In the last few years various synthetic materials have become available which, according to the manufacturers, are suitable for drainage purposes. Their efficiency was examined in test tanks by the Institute for Land and Water Management Research, the IJsselmeerpolders Development Authority and the Heidemaatschappij. Materials that looked promising in the laboratory were used in experimental drainage fields, where they were tested with regard to efficient functioning, permeability and their tendency to silt up.

The Drainage Study and Contact Group recently recommended the use of polystyrene granules in Drakafolie (a type of sheeting) and polypropylene fibre, partly because they are voluminous materials but also because polystyrene granules deposited loose in experimental fields have given very good results in the past, and the industry has been continually urged to deliver pipes prewrapped in this material so that they can be laid mechanically. Polypropylene fibre was developed after it was discovered in North Holland that coconut fibre sometimes rots quickly. The structure of polypropylene fibre is very similar to that of coconut fibre, and it is manufactured in different thicknesses so that the ideal mixture can be composed for use in each particular soil. It is true to say that, owing to the technological developments of recent years, cover and envelope materials have also done a great deal to mechanize and rationalize drainage work.

4. Mechanization of pipe-laying

Until 1954 laying of drains in The Netherlands was carried out entirely by hand. In that year the first "Buckeye" trenching machine was imported from the United States, and within a few years pipe-laying became largely mechanized.

As early as 1958 there were 80 pipe-laying machines operating in The Netherlands. The following figures for land consolidation work on Schouwen illustrate the change:

1954	drainage pipes were laid entirely by hand
1955	20% of pipe-laying was mechanized
1956	75% was mechanized
1957	practically all pipe-laying was mechanized except for some difficult sites
1958	less than 10% was done by hand

The first machines were equipped with wheel excavators, but it was not long before chain excavators began to be produced in The Netherlands. The chain excavator was originally built onto a wheel tractor; later the rear wheels were replaced by caterpillar tracks. Later still, machines were built with a complete track undercarriage. In the course of the years many technical improvements have been made, horsepower has been considerably stepped up and the depth of digging increased.

The first tests with a trenchless machine was carried out in 1968. With thi system the aperture needed for inserting the pipe is not made with a wheel or chain excavator, but with a plough body. Through the rear of the plough a corrugated PVC pipe is deposited on the floor of the mole passage. This method has already been used for some time abroad. Further information abou the development of trenchless drainage may be found in the article entitled "Trenching and trenchless drainage" (Paper 3.07).

In the past few years various automatic depth regulation systems have also been developed which makes it unnecessary to fix the depth of each drain length beforehand. The reliability of these systems has not yet been sufficiently tested; this is a field of investigation for the future. Indeed, an automatic depth recorder must first be developed which can be combined with the automatic depth regulation system already in use.

5. Drainage maintenance

As we said in the introduction, insufficient attention is sometimes paid to drainage maintenance in areas where drainage is new and has been laid in the course of land consolidation. This leads to complaints about and dissatisfac tion with the way in which the drainage network functions. In areas where drains are laid it is therefore essential to provide instruction about maintenance, and interest contractors in buying maintenance machines. In Drenthe for instance, in collaboration with the National Council for Agricultural Development and the Government Service for Land and Water Use, regular demonstrations are given of ditch and drainage maintenance machines. The local land consolidation committee also carries out one maintenance, at the same time checking that the work has been properly executed. As many local contractors as possible are invited to come and watch this maintenance, most of whom immediately purchase the necessary equipment. Flushing equipment with a pressure of 15-20 atm. has also been acquired by farmers, cooperatives and farming communities for their own use. It is essential not only to flush through the drains but also to maintain open dicthes and the drain outlets into the dicthes, Failure to do so causes a blockage or reduces the drain discharge. This is especially the case where the groundwater contains iron. In these regions the end pipes have to be cleaned regularly several times a year. This takes little time and is usually most effective.

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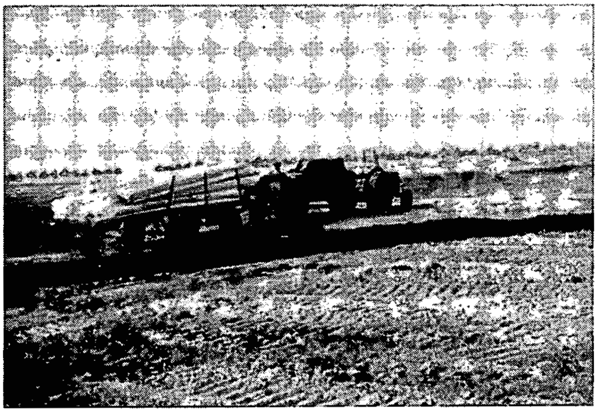

Fig.1. A trailer carrying smooth plastic pipes and another with rolls of peat fibre, drawn by a tractor, make their way along each drain trench.

Fig.2. A store of corrugated PVC pipes.

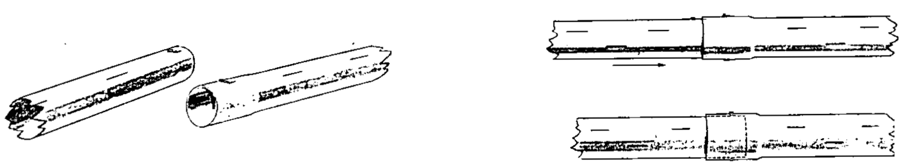

Fig.3. Smooth PVC pipe with solvent-welded joint.

Fig.4. Mechanized drainage with clay pipes. The pipes are taken from the trailer and laid on a platform. Two workmen apply the peat fibre.

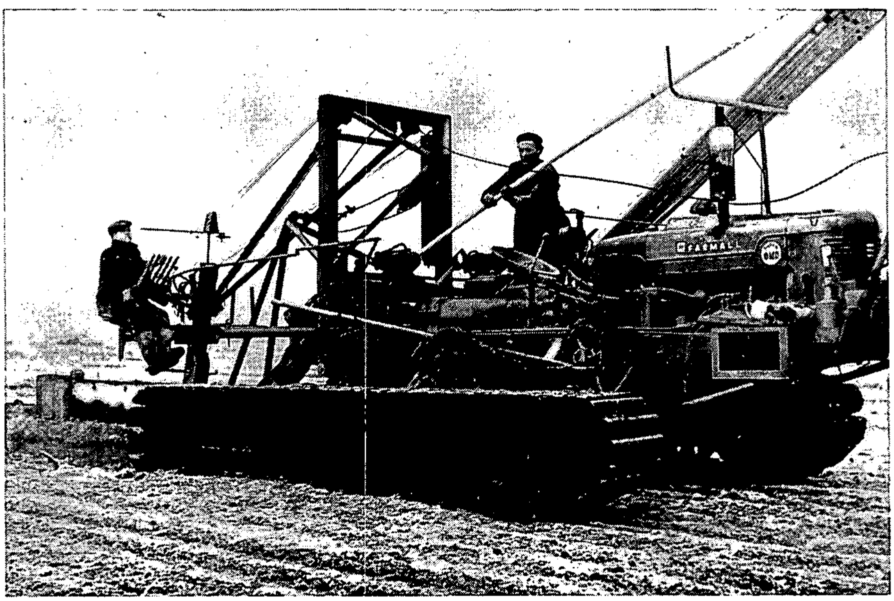

Fig.5. Mechanized laying of smooth plastic pipes pre-wrapped in glass fibre.

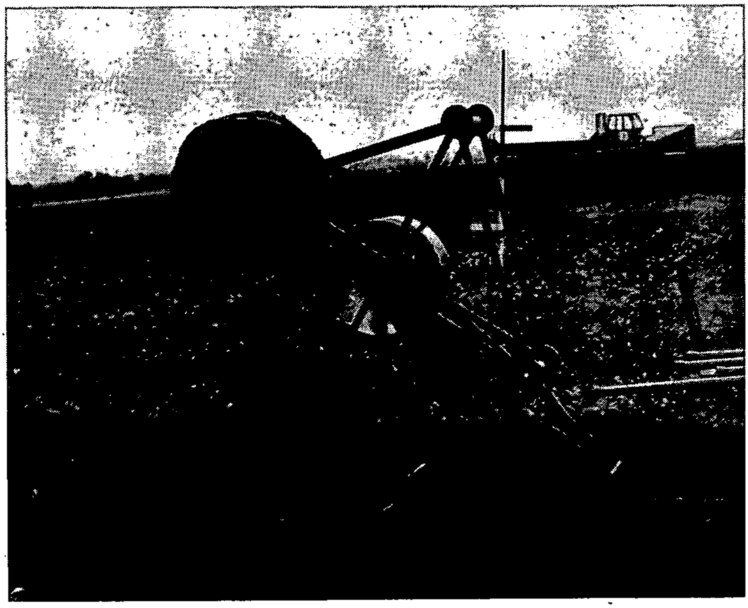

Fig.6. A reel that can hold four rolls of fibre strip.

Fig.7. Covering pipes with flax strip.

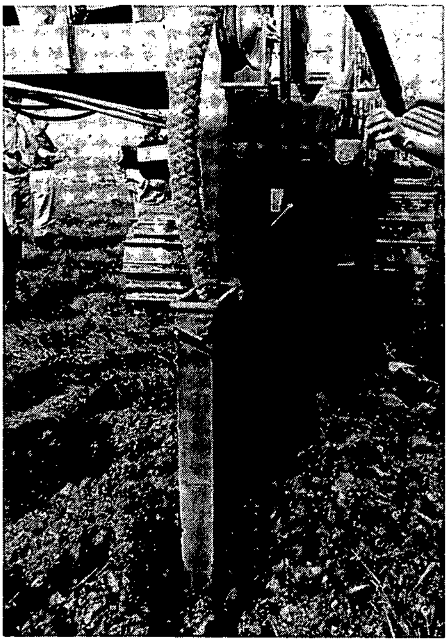

Fig.8. Mechanized laying of pre-wrapped corrugated PVC-pipes.

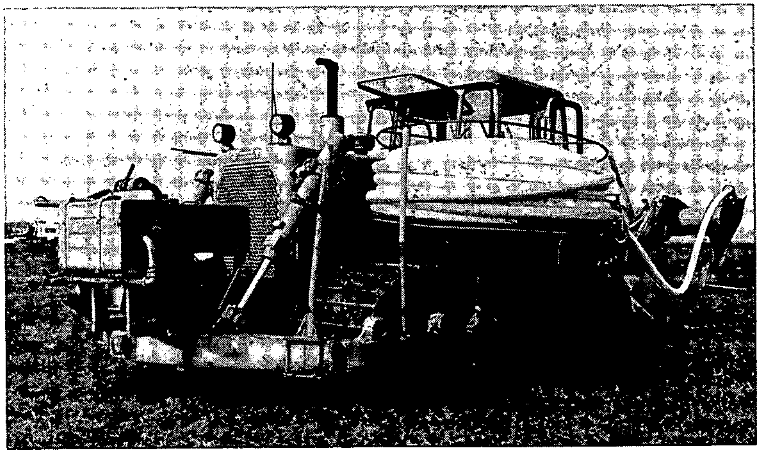

Fig.9. Mechanized pipe-laying with the trenchless machine of "Willner" using PVC-pipes pre-wrapped in polystyrene granules in Drakafolie.

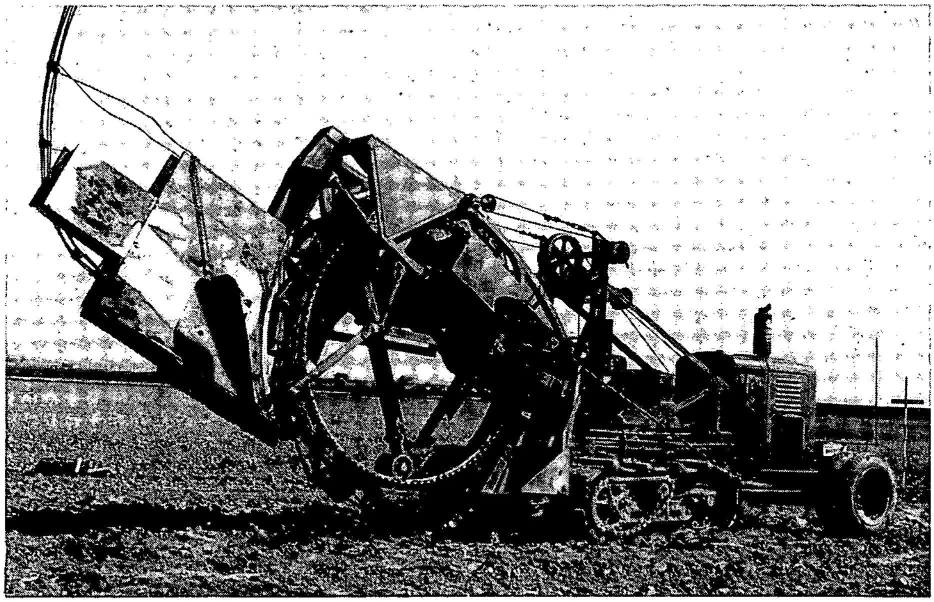

Fig.10. "Buckeye pipe-laying machine.

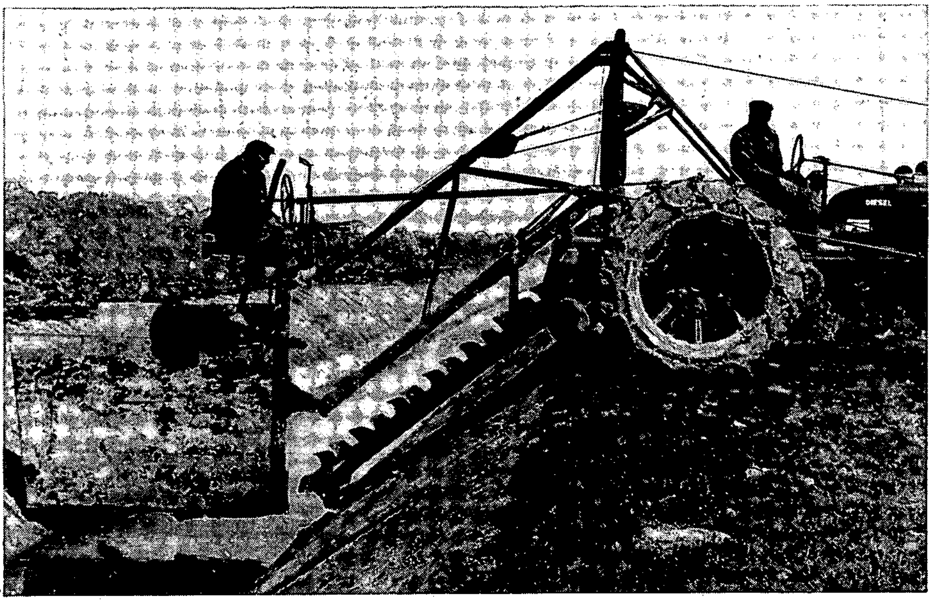

Fig.11. Chain-digging machine built onto a tractor.

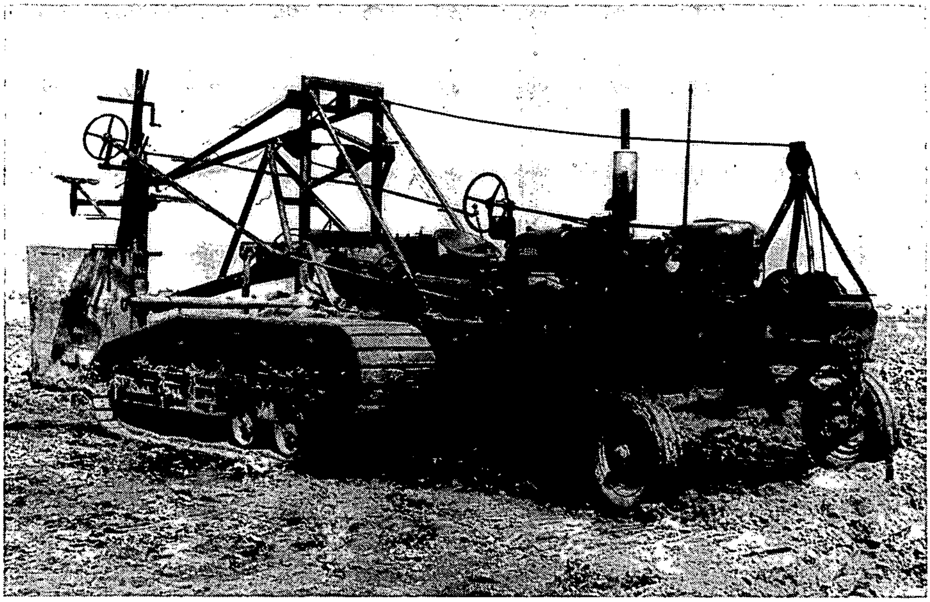

Fig.12. Chain-digging machine with half tracks built onto a tractor.

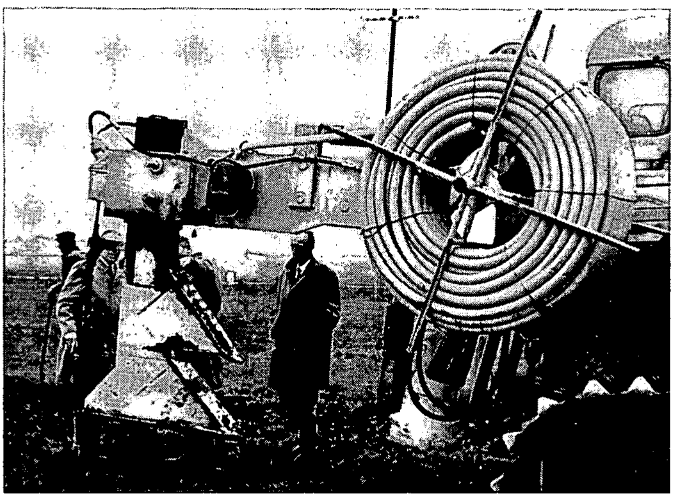

Fig.13. Trenchless drainage being tried out in The Netherlands in 1968.

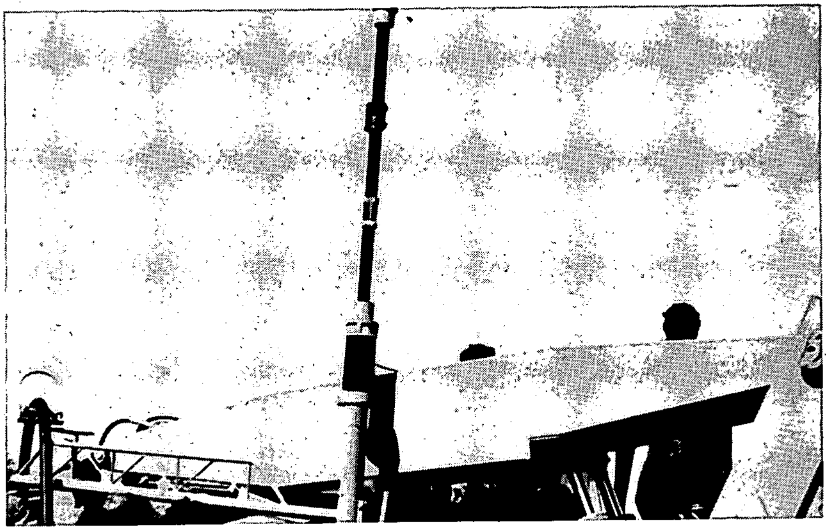

Fig.14. A trenchless drainage machine fitted with an automatic depth regulation system. The photo shows the receiver.

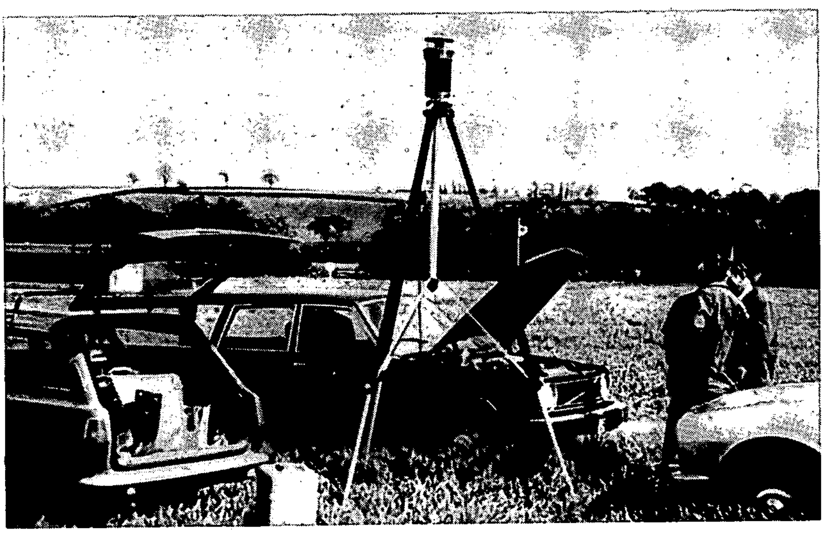

Fig.15. Transmitter of the automatic depth regulation system.

Paper 3.05

TRENCHLESS DRAINAGE EXPERIMENTS IN DENMARK

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Summary

After the appearance of corrugated plastic drain pipes the interest in trenchless drainage has increased, mainly because of greater working capacity of the machinery and consequently smaller constructions costs. In 1972 the Research Department of Hedeselskabet (Danish Land Development Service) initiated experiments with trenchless drainage and investigations of practical drainage projects.

In four experiments trenchless drainage has been compared with trench drainage The experiments were carried out on sandy loam and loam soils at different moisture contents. Two types of machines for trenchless drainage with different shape of the plough body and different depth control system were used.

Introduction

Trenchless drainage carried out as mole drainage has been utilized almost as long as drainage by closed drain systems has been known. The method is based upon a torpedo shaped sword being drawn through the soil by which a soil "mole" is made that functions as a drain during some time. At the beginning the sword was built upon a horse-drawn plough. Today the drawing power is usually a crawler tractor.

The moles have got a limited durability, and in practice the method is used only in some peat soils or in clay soil with more than 40 per cent clay. For soils with little hydraulic conductivity the mole drainage is often carried out at narrow intervals across and above drain pipes with a highly permeable backfill, for instance gravel (Hallgren & Johansson, 1948). This method is only of little interest under Danish soil conditions, but has been utilized to some extent in other countries, for instance in England.

Many methods have been tried to stabilize the moles. Ede (1957) mentions depositing of cement during mole drainage, and Busch (1958) describes utilization of plastic tapes which are formed as a pipe in the mole.

Utilization of trenchless drainage methods was not very important until the appearance of corrugated plastic drain pipes in the early sixties.

Various types and sizes of trenchless drain machines have been developed. The types which are of interest for Danish soil conditions are built as crawler tractors of 100-300 HP with a pipe laying device formed as a torpedo or as a plough or a combination (see Lundegren, 1974). Mainly corrugated plastic pipes are used, but some machine types are able to lay tile pipes. The biggest machines can work at a depth of 1.8 m with a maximum pipe diameter of 15 cm inclusive of pre-wrapped filter materials. The output of work may be very great, about 600-700 m of pipes can be laid per hour in a more or less stone-free soil. It is double as much as the output by chain trench machines. However, as an average over a longer period the output for both machines is much smaller.

By increasing speed the demand for a quick and exact depth control system securing the pipes being laid with an even slope is increasing. One can distinguish between the following systems:

1. Manual control of the hydraulic system by sighting to target rods, placed traditionally by levelling of height piles.
2. Manual radio control of the hydraulic system from sights adjusted for suitable slope of each pipe and a sighting point at the machine.
3. Automatic telecontrol by a halogen light adjusted for suitable slope for each pipe and a receive and control unit at the machine.
4. Automatic telecontrol by laser adjusted for suitable slope of a drainage section and a height adjustable receive and control unit at the machine.

The systems have got different working and functional advantages and disadvantages. With manual systems it takes some time to observe and correct deviations from the projected depth. After hitting stones the machine can be stopped immediately, the obstacle be removed, and then the machine continues at the same sight line. If the stones only cause a small raising of the sword, and if the projected slope is big, one may continue with less or no slope at all, until the original sight line is reached, by which back slope can be avoided. In such a case an automatic depth control might cause back slope, because the sword continues immediately after the stone in the original depth.

As to the manual systems, it is considerably easier to mount telescopes than a system of target rods. System 4 utilizing a plane of laser rays demands a very careful mounting. In order to utilize the laser equipment rationally, the ground must be relatively level, i.e. the difference in height must be only a couple of metres in the area in question.

The three first mentioned depth control systems have been examined by Voss and Zimmerman (1974a, 1974b). They found great deviations between actual and projected drain depths and slopes, especially at the beginning of the pipes. From their results one might expect some back slopes to appear when manual depth control is applied to projected slopes less than approximately 0.2 per cent. In a stone-free soil and by using automatic depth control, it should be possible to obtain a minimum slope of 0.05 per cent without any stretch with back slope.

Materials and methods

In December 1972 Hedeselskabet (Danish Land Development Service) made 3 experiments in order to compare trenchless drainage to chain trenching drainage. In September 1973 another experiment with trenchless drainage versus trenching by back-acting excavators was made. In all the experiments corrugated PVC-pipes with and without prewrapped filter materials were used.

Nr. Økse, Brovst: The land is a post glacial marine deposit, and the soil is sandy loam with a clay content of about 16 per cent (Table 1). The flat ground had a high groundwater table and was used as pasture. The projected drain depth was 1.2 m, the slope 0.2 per cent, the drain length 100 m, and the distance between drains 18 m. The trenchless drainage machine was a "Drainomat" with manual depth control after target rods. The chain trencher was a Holland-drain make.

TABLE 1. Soil texture of the subsoil at the four experimental fields

Particle size	0-0.002	0.002-0.02	0.02-0.2	0.2-2.0 mm
Nr. Økse	16	20	63	1
Viumgard	18	10	39	33
Østrup	14	13	50	23
Frederiksdal	21	21	37	21

Viumgard, Ulfborg: The land is sloping, and the soil is a sandy moraine loam with a clay content of about 18 per cent. The experiment was laid out in a rainy period, and the autumn ploughed soil had a high moisture content to below drain depth. The projected drain depth was 1.2 m, the slope 0.5 per cent, the drain length 100 m, and the distance 18 m. The drainage machines were the same as at Nr. Økse. In order to obtain the projected drain depth, the trenchless drainage machine had to be drawn by an extra crawler tractor.

Østrup, Hornslet: The land is undulating, and the soil is a moraine loam with a clay content of about 14 per cent. The experiment was laid out when the moisture content was near field capacity. The projected drain depth was 1.2 m, the slope 1-2 per cent, the drain length 100 m, and the distance 18 m. The drainage machines were the same as at Nr. Økse. To steer the trenchless drainage machine transverse to the natural slope and to obtain the projected drain depth, the machine had to be drawn by a four-wheel driven tractor.

Frederiksdal, Nakskvov: The land is flat, and the loamy moraine soil has a cl content of about 21 per cent. At the time of drainage the soil was dried up by the previous crop. The trenchless drainage machine was a "Cornelius" with radio control of the drain depth. Trenching was done by a back-actor. In order to obtain the projected drain depth of 1.1 m, the trenchless drainage machine first had to grub to half drain depth, and then do the final run and laying of the pipes. The projected slope was 0.4 per cent, the pipe length 100 m, and the distance 12-16 m.

The drainage efficiency of each drain was estimated on the basis of simultaneous measurements of drain discharge and groundwater levels at a distance of 0.75 m from the pipe centre. In most cases there is a rectilinear relation between the head of water at a certain distance from the pipe and the discharge (Luthin & Worstell, 1959). For a homogeneous soil this slope can be used to compare the drainage efficiency of the different drain pipes in the experiment.

In the experiments different pipe diameters and envelope materials were used, but in this article only results related to the drainage machines will be discussed.

Results and discussion

On the sloping and newly ploughed areas at Viumgard and Østrup the trenchless drainage was not satisfactory in spite of the extra drawing power. It was found that the trenchless pipes were laid 5-10 cm lower than projected, and that stretches with back slope occurred in most of the drains. On the flat pastures at Nr. Økse the depth control was satisfactory except when passing furrows and similar irregularities. There were no difficulties as regards the depth control of the chain trench machine in the 3 experiments. As the control method was the same for the 2 machines and both staffs were experienced, the bad results of the trenchless drainage may be due to a faster laying speed. The measurements at drains with back slopes are excluded from the evaluation of efficiencies.

The experiment at Frederiksdal with manual radio control of the trenchless drainage machine had an entirely satisfactory drain depth and slope.

Corrugated plastic drain pipes with and without envelope materials could be used without any difficulties by the tested machine types.

In Table 2 the average figures of the water level and discharge measurements made are shown together with the calculated efficiencies for pipes laid by trench and trenchless machines. The highest values indicate the best drainage effect. The discharge figures are based on intermittent measurements during runoff periods and do not give information on the total yearly discharge.

Table 2. The average efficiency of drain pipes. Trench versus trenchless drainage.

	number of drains		number of measurements		discharge, l/s·ha		water level, head in m		efficiency l/s·ha·m	
	trenching	trenchless	trenching	trenchless	trenching	trenchless	trenching	trenchless	trenching	trenchless
Nr. Økse										
1972/73	10	10	20	20	0.44	0.21	0.51	0.61	0.9	0.3
1973/74	10	10	40	40	0.50	0.19	0.19	0.27	2.6	1.7
1974/75	10	10	40	40	0.27	0.28	0.16	0.23	1.8	1.2
1975/76	10	10	20	20	0.26	0.33	0.11	0.18	2.4	1.8
1976/77	9	10	36	40	0.42	0.50	0.16	0.20	2.6	2.5
1977/78	9	10	27	30	0.28	0.36	0.13	0.19	2.2	1.9
Viumgaard:										
1972/73	6	6	24	23	0.23	0.20	0.29	0.37	0.8	0.5
1973/74	6	6	42	42	0.49	0.37	0.47	0.51	1.0	0.7
1974/75	6	6	42	42	0.51	0.39	0.49	0.54	1.0	0.7
1975/76	6	6	36	36	0.25	0.25	0.31	0.42	0.8	0.6
1976/77	6	6	18	18	0.37	0.33	0.37	0.47	1.0	0.7
1977/78	6	6	66	66	0.33	0.32	0.34	0.39	1.0	0.8
Østrup:										
1973/74	3	3	6	6	0.22	0.15	0.40	0.31	0.5	0.5
1974/75	3	3	21	21	0.38	0.27	0.39	0.32	1.0	0.8
1976/77	3	3	6	6	0.14	0.09	0.28	0.14	0.5	0.6
1977/78	3	3	16	16	0.28	0.20	0.32	0.24	0.9	0.8
Frederiksdal:										
1973/74	6	6	11	16	0.26	0.34	0.25	0.31	1.0	1.1
1974/75	6	6	13	16	0.37	0.41	0.27	0.37	1.4	1.1

The table shows that at Nr. Økse the drainage efficiency was relatively low as regards the trenchless method in the first year after drainage. Apparently, an increase of the efficiency has happened for both methods, which may be due to a gradual improvement of the soil structure. Further the results indicate that the difference between the efficiency of the two drainage methods is being reduced as time goes by. The efficiency of the trenchless method is about 80 per cent of chain trenching in the last part of the period. The same relation of efficiencies was found in the experiments at Viumgard. At Østrup the results indicate that the efficiency of the trenchless drainage is almost as good as the efficiency of chain trenching. At Frederiksdal the efficiency is the same for both drainage methods.

The experiments show that if the trenchless drainage is made when the soil is dried up to drain depth - i.e. normally in the period of June to October - a drain effect just as good as by trench drainage can be obtained at a moraine loam soil. If the trenchless drainage method is used when the soil is wet to drain depth, a lower drain effect must be expected. The reason is that the soil loosening effect of the sword at a given type of soil and a given depth primarily depends upon the water contents of the soil. Under dry conditions the soil is partly lifted, cracks are formed, and after laying the pipe some loose earth falls into the "grub track". With a high moisture content in the soil the sword will press the soil sideways and compact it, and after laying the pipe, the furrow will close again. This will decrease the hydraulic conductivity of the soil. However, the experiments indicate that the soil structure and thus the conductivity will improve later on.

Conclusions

The present knowledge of the applicability of trenchless drainage under Danish soil conditions can be summarized as follows:

The trenchless drainage system is most suited for relatively flat areas. For undulating areas drainage transverse to the natural slope may give difficulties in machine steering. Irregularly undulating areas together with limited drain depth may restrict the use of rational drainage schemes.

A disadvantage of the trenchless method is that damage to existing drainage systems will not be observed immediately, but may result in wet spots later on. At this time searching and repair can be difficult and expensive.

The experimental results have shown that trenchless drainage carried out on a loamy soil at low moisture content results in a satisfactory drain effect. Under wet soil conditions the resulting drain effect was smaller than by normal trenching. The best time for drainage will be in the period of June to October.

On loamy sand trenchless machines are less sensitive to stones than trenching machines.

On sandy soil it can be assumed that the trenchless method results in satisfactory drain effect irrespective of the soil moisture content at the time of drainage.

The trenchless method does not exclude using pre-wrapped pipes to prevent intrusion of silt and sand.

Practical experience has shown that on heavy, badly structured, clay soil the trenchless method results in an unsatisfactory drain effect.

On mineral soil with low moisture content the power required to pull the sword through the soil often impedes a drain depth to more than 100 cm without previous loosening of the soil to about two-thirds of the actual drain depth. The additional soil loosening is always advisable as it might increase the drain effect.

On flat areas and by using levelling sight and manual depth control a minimum slope of 0.2 per cent can be achieved satisfactorily. Passing furrows, etc. at a normal speed may cause unacceptable depth control.

As the trenchless machines can cover about 6 hectares a day, and moving to the next area is time-consuming, the method is best suited for areas above this size.

Compared with traditional trenching, control with the trenchless drainage is both difficult and time-consuming. The pipes can, however, be located by a special probe rod immediately after laying.

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Paper 3.06

COMPARISONS BETWEEN TRENCHLESS AND TRENCHING SUBDRAINAGE

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Summary

First pedological and hydrological field researches have shown, that in a marsh soil (heavy silty clay) with groundwater influence the trenchless subdrainage method has somewhat better effects than the trenching subdrainage machine with cutter chain, especially if they work under wet conditions.

Introduction

Subdrainage is still today an important way of soil melioration in Central Europe. Since 1955 an area of 60,000 to 70,000 ha have been drained yearly in the Federal Republic of Germany, total more than 1.4 mill. ha (Eggelsman, 1978).

For about five years more and more trenchless subdrainage machines are working in our country, in North-western Germany nearly 70 machines. This machine type needs only 2 to 3 men for the fully mechanized drainage work in the field (marking-off, levelling, laying corrugated plastic pipe).

Problem

The German drainage contractors prefer the trenchless machines, because this method is very economical. From the view of soil science we recommend the trenchless method for peat, marsh and loam soils, because these soil types in our lowland areas are mostly wet and soft nearly the whole year (influence of groundwater and humid climate).

In these wet soils the trenching subdrainage machine destroys the soil structure with its cutter chain. Wet disturbed soils, refilled into the drain trench, prevent or hinder the water streaming to the drain pipe (Fig.1). In that case we find a high entrance resistance of groundwater at the drain (Cavelaars, 1967; Eggelsman, 1969).

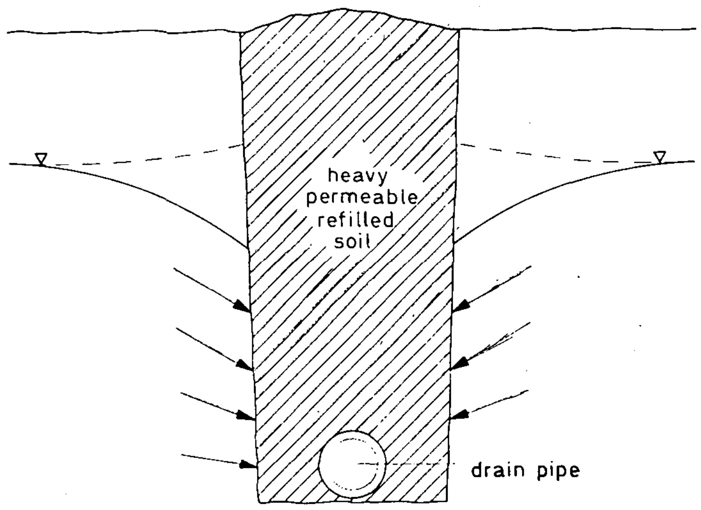

Fig.1. Scheme of the groundwater level at drain trench after refilling with wet silty clay soil.

The trenchless subdrainage machine breaks up the soil (Fig.2). This machine is careful with the soil structure, if the plough has the right form and angle. If the soil is dry enough, it even meliorates the soil structure (Eggelsman, 1973).

But in fact there are no exact researches upon the influence of the trenchless subdrainage machines on the soil structure and the drainage effect.

Fig.2. Trenchless drain. The trenchless drainage machine broke up the soil layer about 35 cm.

However, there are remarks from colleagues in The Netherlands on soil pressure and soil smearing by this machine type, which means that the drain distance by trenchless subdrains should be about 10% smaller than by trenching drains (personal informations, see also Naarding, 1976).

Locality

Here we give an account of measurements at two drainage fields, laying in the marshland near Bremerhaven. The soil surface above sea level is about +1 to +1.5 m N.N., the drainage goes into the new Grauwall Channel, which flows via a sluice to the river Weser. The position of both fields to the main drain is equal. The mean annual rainfall is 740 mm.

Both machine types worked in 1972 under substantial same conditions. The drainage fields have a heavy silty clay, type: Brack-Marsch (see Table 1).

TABLE 1. Dates of soil analysis from 90÷120 cm depth

Analysis		Dimension	Plot trenchless	Subdrainage trenching
fine sand	> 60 µm		3	2
coarse silt	60 - 20 µm	%	41	34
medium + fine silt	20 - 2 µm	of dry matter	23	32
clay	< 2 µm	content	33	32
organic matter		%	2.5	2.5
$CaCO_3$		%	0.05	0.43
pH-value (nKCl)		-	4.6	5.2

The soil has a hydraulic conductivity of 15 cm/d (=upper layer) and 30 cm/d (layer under drain).

Before drainage both fields were used as grassland, after subdrainage the farmers used them as arable land, mostly with cereals and rape.

The single subdrainages have a depth of 1.3 m (at the outlet) to 0.9 m at the end. The (artificial) drain slope is 0.25%. The drain distance is 10 m. The corrugated plastic pipe (PVC) have coco-filters (full-wrapped filter pipes).

Field research method

In the winter 1977/78 we measured in both drainage fields:

- depth of groundwater under surface between the drain pipes,
- potential height of groundwater at drain pipe,
- drain water runoff,

- soil moisture and consistence through estimation by finger touch (Müller et al., 1970),
- soil infiltration rate above drain, measured at a depth of 50 cm under surface,
- soil pressure resistance in the former drain trench resp. direct upon or near the drain pipe,
- soil shear resistance direct upon or near the drain pipe,
- soil condition of the surface layer.

All pedohydrological field methods are described by Eggelsman (1977).

Results

The soil moisture and soil consistence in the former drain trench resp. direct above the trenchless drain, estimated in the soil layer of 40-80 cm under surface are as in Table 2.

TABLE 2. Estimation of soil moisture and consistence

Plot of subdrainage	Soil moisture	Soil consistence	pF-value
trenchless	moist	stiff to plastic	3.0 - 2.5
trenching	wet	soft	2.2 - 1.4

The drain water runoff from both kinds of subdrainage fields was nearly the same.

In both kinds of subdrains we could see the beginning of iron ochre clogging at the drain outlets.

The other results of the pedohydrological field measurements are summarized in Table 3:

TABLE 3. Results of the pedohydrological field measurements

Kind of measurement	Dimension	Plot of trenchless	Subdrainage trenching
Groundwater depth in the centre between drain pipes	cm	63	76
Potential height of groundwater at the drain pipe	cm	14	26
Rate of soil infiltration	cm/d	65 (medium)	600 (very high)
Soil pressure resistance	kg/cm^2	4-6 (medium)	1-2 (very slow)
Soil shear resistance	kg/cm^2	0.5 (medium)	0.2 - 0.1 (slow to very slow)
Soil condition of the surface layer	-	satisfactory	moderate

Discussion

Our first researches in one soil type - certainly in a difficult and sensitive marsh soil - show that the pedohydrological situation at the trenchless subdrainage field may be somewhat better than at the trenching subdrainage field. Also the soil conditions seem to be somewhat more favourable at the trenchless subdrainage field than at the trenching subdrainage field (Tab.3).

The contrasts between infiltration rates and soil pressure resistances are caused by the different soil moisture resp. consistence (Tab.2).

Conclusion

Our first test has shown that in silty clay marsh soil the trenchless subdrainage seems to be somewhat better. But more researches are necessary, on the one hand pedohydrological researches over a longer time, on the other hand especially more researches in other soil types.

But it seems that in silty and clay gley soils our opinion is confirmed, that the trenchless subdrainage may be better than the trenching subdrainage system, especially when the machine works in wet soil conditions.

The German subdrainage Standard DIN 1185 (1973) demands that subdrainage works must be carried out only in soils which are dry enough, but that demand is not always attainable in marsh and gley soils in our humid climate (Renger et al., 1975).

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Paper 3.07

TRENCHING AND TRENCHLESS DRAINAGE

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Summary

Draining is often done in The Netherlands using trenching machines: that is to say, a trench of the right depth is dug with a chain or wheel excavator and the drain pipe is laid on the floor of the trench. With trenchless drainage a tunnel is made at the depth at which the drain pipe is to be laid, and the drainage pipe is placed in the ground through an opening at the rear of the plough body.

In recent years trenchless drainage has been practised in different degrees in Europe. We therefore decided to investigate what use various European countries make of trenchless drainage machines, what experiences they have had with them and what research results are available on the efficient functioning of drains that have been installed without trenches. On the basis of the thus obtained data a brief survey will be given on:

Developments in drainage techniques, the most important facts about trenching machines and trenchless machines; the use of trenchless drainage in certain European countries, construction costs of single drainage-system in The Netherlands with trenching machines and trenchless machines, drainage effect of systems constructed by trenching or trenchless techniques.

Finally of both drainage techniques, advantages and disadvantages are summarized.

Introduction

In former times drain pipes were laid in The Netherlands by digging a trench by hand and laying the drainage pipes - also manually - on the bottom of the trench. After a sort of horse-drawn scraper, called *Pratt's Ditch Digger*, had been used, the discovery was made in America round about 1890 that a trench could be dug by means of a wheel. The first machine was called *"Buckeye"*.

In 1954 a "Buckeye" machine was introduced in The Netherlands that could dig a trench to the full required depth, the depth being regulated by means of the sole. The pipes still had to be laid manually. The disadvantage of the digging-wheel machine was, that it had a comparatively small laying capacity When chain excavators were developed, laying capacity was increased.

In the past 20 years the laying of drains has thus been transformed from a completely manual to a more or less fully mechanized operation. The change over from clay pipes via smooth plastic drainage pipes to corrugated plastic pipes has largely contributed to the increased laying capacity of drainage machines. Besides, the laying capacity has also been significantly enlarged in recent years by increasing the tractive power of the machines, resulting in higher digging and driving speeds.

An entirely different development in East Germany is the system produced by mole drainage. The most important disadvantage of mole drainage is the fact that the tunnels are liable to cave in, particularly in less stable soils and when there has been water in them for some time. It was believed that this danger could be eliminated by introducing a pipe into the earth without digging a trench. A mole body, fixed to a vertical knife, was first used to make a tunnel in the ground by pushing the earth aside. The disadvantage of this method was that the earth around the pipe was extremely compacted.

In the course of the years various manufacturers brought out their own models their aim being to reduce the pulling force as much as possible in order to obtain a good laying speed. It was supposed that this could be done by using a not too wide ripperleg with a sharp front. The leg was positioned at an angle of 90° to the horizontal (that is, the bottom of the trench).

From experience with subsoilers we know that this shape demands a rather high tractive power because the horizontal forces are great. Consequently, the earth is pushed aside and therefore not lifted sufficiently. This may cause the soil to be deformed.

Having discovered this, manufacturers tried to find a system whereby the horizontal forces would be reduced and the vertical forces increased. To achieve this, the front of the ripperleg was made more concave, and a flat plate, wider than the rest of the ripperleg and the trench box, was made to extend over the whole length. The whole was positioned at a certain angle to the horizontal.

The advantage is that the earth can be lifted more efficiently down to a certain depth. Generally speaking, the extent to which the earth is lifted depends on the width of the ripperleg and its position to the horizontal.

According to Mr. Gordon Spoor of the National College of Agricultural Engineering in Silsoe, Great Britain, a ripper body with a width of 10-15 cm positioned at an angle of about 45^{o} can still lift the earth down to about 1.50 m below the surface; a ripper body with a width of 20 cm can do so down to about 1.50 m. If these depths are exceeded, the earth below this level will be compacted because the ground will choose the way of least resistance. Spoor calls the point at which the earth ceases to be lifted and starts to slide along the ripper body "the critical depth".

The "Willner" is based on another system. It consists of a V-shaped plough body carried by a crawler tractor. The pipe is inserted in the ground through an opening in the plough body. As the "Willner" moves along, the plough body lifts the earth by about 20 cm, the pipe is laid and the earth falls back and covers it. The depth of the body is regulated hydraulically.

The "Willner" is an improvement on the other plough bodies we have described, in that the horizontal forces are comparatively weak and the vertical forces stronger. This means, however, that it cuts less deep though the ground is sufficiently lifted.

With the results from these different shapes in mind, the Georg August Universität of Göttingen, West Germany, developed the Y-shape, which combines the V-shaped part with the narrow bottom part in such a way as to enable it to reach a greater depth than the "Willner". According to the designers, it achieves an ideal balance of forces, combining minimum horizontal forces with maximum vertical forces. The machine is being used in the West German "Meliorationverband Norden" (Land Improvement Plan "Norden").

Trenchless drainage techniques were developed because it was believed that trenchless machines with their higher speed and lower maintenance costs, could do the work more cheaply than trenching machines. Designers were especially intent on achieving a higher laying speed.

Some general characteristics of the machines

Trenching machines

1. Over the years the power of trenching machines has been considerably increased. About ten years ago most machines were still fitted with motors of 100-150 hp; now, partly under the influence of the trenchless techniques, more powerful motors of up to 200 hp are used. These of course have a greater laying capacity, thus reducing the capacity differences with trenchless machines.

2. Machines fitted with caterpillar tracks and able to achieve a depth of 1.80 m vary in weight from 9.5 to 15.5 tons. The ground pressure ranges from 0.24 to 0.29 kg/cm^2 ($0.24 \times 9.81\ N/1 \times 10^{-4} m^2$ and $0.29 \times 9.81\ N/1 \times 10^{-4} m^2$ respectively).

3. Most machines have a maximum working depth of 1.80 m. Those built for laying main drains, horizontal dewatering systems and deep drains in urban areas can generally reach a maximum depth of 3.00 m.

4. Laying capacity depends on the depth and length of the drain, the soil structure, soil and site factors and the power of the machine.

Good organization is also essential for achieving maximum capacities; delays caused by poor organization have a direct effect on performance.

Trenchless machines

1. Machines vary widely in horse power: viz. between 140 and 320 hp. Generally speaking, the power increases according to the depth to be reached.

The power of the machine also depends, of course, on the shape of the plough body.

According to Trafford, the increase in horse power required is represented approximately by the formula ($P = XD^3$). P being power, D depth and X a constant depending on the shape of the plough body.

Trafford considers trenchless drainage justifiable to a depth of 80-100 cm; greater depths require more horse power and also involve greater risks. Some machines are therefore fitted with a winch to compensate for any lack of horse power. A number of unsophisticated machines are also towed by a winch on a tractor.

2. Machines can be divided roughly into two classes: the first ranges from 13 to 23 tons and has a ground pressure of between 0.24 and 0.29 kg/cm^2 and the other rangers from 27 to 41 tons and has a ground pressure of between 0.46 and 0.72 kg/cm^2. Soil factors (i.e. the permissible ground pressure per cm^2) are of great importance when choosing which weight to use.

3. The maximum depth which can be reached is between 1.20 and 2.20 m. This can only be achieved under ideal conditions: i.e. a more or less dry topsoil and a moist subsoil. The topsoil must be dry enough for the tracks to get a grip in order to exert the necessary pulling force.

4. The laying capacity on certain factors, some of which we mentioned in connection with trenching machines: the depth and length of the drains, the soil structure, soil and site factors, the topography, the shape of the plough and the weight and power of the machine. These factors determine the maximum laying capacity, together with the conditions mentioned under 3: a more or less dry topsoil and a moist subsoil, and a not excessive depth. If the topsoil is wet, capacity will be swiftly reduced and will hardly exceed - or may be less than - that of the trenching machines.

5. Trenchless machines can be used most profitably on sites of over 10 hectares because the loading and transportation of these heavy machines is more time consuming, requires heavier low loaders and therefore, higher costs.

A trenching machine and trenchless machine combined

It is not always possible or advisable to use trenchless machines. On wet topsoil, for instance, their capacity is severely reduced, and when laying very deep drains they require excessive pulling force or exceed the critical depth. To overcome these difficulties, a drainage machine made by the firm of Steenbergen in Klaaswaal has been adapted in such a way that it can be fitted with both a digging chain, as well as with a ripper body.

If this combination model is purchased, the motor unit will have to be built as a tractive tool. The additional digging chain and ripper body units can be changed fairly easily. The construction of this machine is such that it can be used in a greater number of situations.

Application of trenchless drainage in some European countries

Notwithstanding the fact that trenchless techniques were introduced more than ten years ago, one could describe their development as slow and hesitant, as against the development in West Germany where particularly in the northern alluvial soil over 50% is constructed in trenchless systems.

In France 15% were laid by trenchless machines in 1974 and 30% in 1975. In England and Wales it was 10%, in Austria 5% and in The Netherlands also 5%.

Cost of laying under drainage with trenching machines and trenchless machines

The cost of drainage machines per meter depends on the purchase price of the machines, depreciation, interest, insurance, fuel consumption, lubricants, replacement chains, knives and other wearing parts, depreciation of tracks and other repair costs.

From the calculations it has appeared that the machine costs per meter, at the same combinations of hourly and yearly capacities, are 10-20% lower when trenchless machines are used. This advantage is mainly gained by the fact that trenchless machines have less wearing parts than trenchers. For trenchless machines repairs are confined to regular welding of the point of the ripper leg.

Because of the fact that in case of trenchless drainage, the machine is a drawer instead of a carrier, the track wheels and drive shafts have more to stand.

Trenchless machines consume on average 10-15% less fuel than trenching machines. Under favourable laying conditions they also have a greater hourly capacity, which means that they cost less per meter. The total cost of drainage per meter is made up of the cost of the machine, the materials and the tractor, wages, overhead supervision costs and taxes. Calculations have shown that trenching machines with a yearly capacity of 500 km and hourly capacities of 200 and 400 m, using corrugated pipes instead of clay ones, could save 0.56 and 0.25 guilders per meter respectively, i.e. 17% and 9% on the total costs.

When the costs of a trenching machine was compared with that of a trenchless machine of the same hourly capacity, using PVC corrugated pipes, the trenchless machine was found to save 0.31 and 0.16 guilders, or 11% and 8% respectively on the total costs.

The greater saving was therefore made by changing from clay pipes to corrugated PVC pipes. The differences become more pronounced of course, if one type of machine can achieve a greater hourly capacity than the other. If a trenching machine with a yearly capacity of 500 km and an hourly laying capacity of 300 m of corrugated PVC pipe is compared with a trenchless machine whose capacity is 400 m per hour, the latter will be found to save 0.40 guilders per meter, or 17% of the total costs. Both calculations assume ideal working conditions.

The functioning of drains laid with trenching machines and trenchless machines

The effect of drainage is largely dependent on the degree of permeability of the soil close to the drains. Particularly if they have been laid with a trenchless machine, it depends whether or not this ground has been deformed or compressed, and to what extent an open structure has been created between the drain and the surface: in other words, to what extent the ground has been broken up. This, as we have already pointed out, is determined partly by the shape of the plough body.

If the ground around the drain was deformed or compressed when the drain was laid, the question is how long it will take for the structure and the permeability to return to normal.

It should also be remembered that when drains are laid with a digging chain, the trench walls and the excavated ground also suffer structural damage. This damage will be all the greater if the drains are laid in wet conditions (high groundwater levels).

With higher digging speeds, the risk of the soil getting deformed or compacted, will also increase; this applies particularly to wet plastic clay soils.

Experience in other countries

Generally speaking, little is known about the comparative efficiency of drains laid with the trenchless machines and those laid with trenching machines. Opinions differ greatly from one country to another, which is understandable in view of the different conditions under which drainage is carried out.

In Great Britain, where drains are generally laid in heavy soils, the drainage system is a combination of moling and subsoiling. There is in fact a vertical discharge to the drain whereby low permeability of the trench wall does not impair the efficient functioning of the drain.

In France drains are often laid in stony soils. As this causes a high degree of wear and tear to knives and chains, trenchless drainage is regarded as an economical solution for this type of soil.

In Austria, the "Bundesanstalt für Kulturtechnik und Bodenwasserhaushalt" (Federal Institute of Agricultural Engineering and Groundwater Management) at Petzenkirchen carried out a number of excavations on plots of ground where the drains were not functioning satisfactorily. From the resulting data it was concluded that - as with other methods - there are limits to the use of trenchless drainage. As far as one can see from the few investigations, these limits were due partly to mechanical factors and partly to the soil. There were sufficient findings to show that trenchless drainage produced unsatisfactory results in plastic soils (with consistency limits, according to Atterberg, of between 50 and 80). In other types of soil few difficulties were encountered, provided the drains were not laid too deep.

In Germany experiments were carried out in 1971 under favourable conditions in the "Großbüttel" experimental drainage field in Schleswig-Holstein. The investigation showed that resistance to the flow of water to the drain decreased in time, which confirms an investigation in the IJsselmeer Polders. It should be mentioned that in both Großbüttel and the IJsselmeer Polders the maturity of the soil was an important factor. It was also found that in both the case of drains laid by trenching machines and of those laid by trenchless machines, the initial resistance was high, but had decreased by two-thirds a year later.

When drains are laid, the soil in their immediate vicinity is sometimes violently disturbed, which impairs permeability. This gradually improves. This is shown in the table below.

Permeability of the soil in the immediate vicinity of the drain in relation to time.

Drainage system	Permeability in m/day *					
	Nov. '71	Mar. '72	Apr. '72	Nov. '72	Febr. '73	Apr. '75
Trenchless PVC without a filter	0.034	0.049	0.065	0.108	0.142	0.198
Trenchless PVC with a filter	0.043	0.056	0.076	0.120	0.173	0.217
Digging wheel; clay pipes of 33 cm long	0.106	0.142	0.184	0.284	0.384	0.394
Digging wheel; clay pipes with a length of 50 cm	0.143	0.188	0.246	0.376	0.468	0.457

* *To convert to the SI system (μm/s), multiply the permeability values by 11.6*

Permeability improved considerably over the years, but after trenchless drainage with the Nordhastedter Dränpflug, the soil never regained anything like the degrees of permeability it had before it was disturbed, which was 0.5 m/day.

Experience in other countries shows that it is impossible to draw general conclusions because the conditions under which drainage is carried out vary so much from one country to another.

Experience in The Netherlands

During the last few years various investigations have been conducted in experimental drainage fields in The Netherlands with two objectives: to compare drains laid by trenchless and trenching techniques, and to try out various cover materials.

- These investigations show that trenchless drainage in certain kinds of soils causes more compression and deformation of the soil than trenching drainage. The soil recovers eventually, but it is a slow process. The use of trenchless drainage therefore involves a considerable risk for the land user.
- In experimental fields of clay and heavy sandy clay soil in the IJsselmeer Polders, drains laid with a trenchless machine were found to have a much lower discharge the first winter after being laid than drains laid with a trenching machine. This was also the case in subsequent years, although the disparity gradually decreased.

- In 1973 drains were laid 135 cm deep with the trenchless machine in Friesland. The soil structure was 30 cm light sandy clay on 100 cm heavy sandy clay on light sandy clay to sand.

  The first winter the discharge from the trenchless drains was about half that from the trenching drains. The reason given was that the soil had been deformed by the plough body. It was also found that the drains laid with trenchless machines had silted up more, which was ascribed to piping: i.e. an excessive vertical discharge of rainfall through the incision to the drain.

  This can be prevented by ramming the top soil down thoroughly. The second winter the differences were more or less the same.

- In investigations held in experimental drainage fields in Drenthe, where the soil was composed of peat on sand, no clear differences could be found between trenchless and trenching drainage. The drains were laid in the sandy subsoil.

Comparison of the advantages and disadvantages of drainage with trenchless and with trenching chain machines

- Under favourable conditions a trenchless machine laying corrugated PVC pipes at a depth of between 0.90 and 1.20 m can achieve a considerably higher capacity than a trenching machine.
- Drainage can be continued longer in spring with less damage to crops. This has the added advantage that drainage is carried out while the groundwater level is falling.
- Generally speaking, the number of days a trenchless machine operates under favourable conditions (dry topsoil and moist subsoil) are fewer than with a trenching machine.
- If drains are laid very deep, the trenchless machine needs a great deal of extra pulling force, and there is a greater risk of the soil around the drain being compressed and deformed.
- In wet conditions trenchless machines damage the structure of the topsoil much more than trenching machines, and their laying capacity is severely reduced.
- In hard, stony soils the digging chain machine has the disadvantage that the chain and knife are very subject to wear and tear. This can be partly remedied by cutting the earth first with a ripper. The trenchless machine has a definite advantage in this respect.
- Re-instatement work after trenchless drainage should not be neglected, especially if the plough lifts the earth well. Where trenching machines are used the earth can easily be put back with a scraper/grader or a rammer.
- It is difficult to check the depth of drains laid with a trenchless machine.

- In regions where a porous fill material is used (for instance in Great Britain), a considerable saving can be made in this material with trenchless drainage because the annular space is considerably smaller.
- In smaller areas (up to 10 ha) it is not worth using trenchless machines as the ratio of hours worked to transport costs is not favourable. This does not apply in the case of simple machines.
- Crossing recently filled ditches often causes problems when a trenchless machine is being used, as the caterpillar tracks have insufficient grip on the loose ground.

Conclusions

- Trenchless drainage is a good solution in stony soils, since the chains and knives of a digging chain machine are subject to excessive wear and tear.
- Fewer hours on average can be worked with the trenchless machine since it can only be operated when the topsoil is reasonably dry.
- When drains are laid at considerable depth, the necessary pulling force rises excessively ($P = XD^3$).
- If the critical depth is exceeded, the soil around the drain is extremely compressed or deformed.
- The critical depth depends on the shape of the plough body.
- As regards the functioning of the drains, trenchless drainage is suitable for sandy soils but cannot as yet be recommended for sandy clay and clay.
- All the factors involved should be carefully considered before a trenchless machine is chosen. It should be borne in mind that its uses are, on the whole, more limited than those of a trenching machine.

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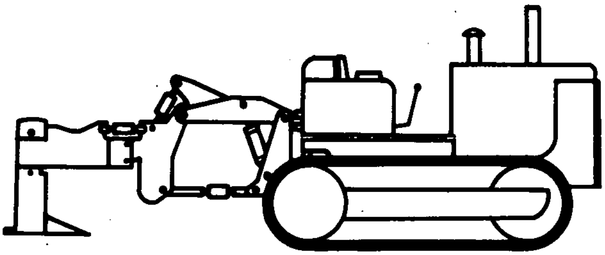


Fig.1. Fairly narrow ripper-leg with sharp front edge. The ripperleg is positioned at an angle of 90^0 to the horizontal.

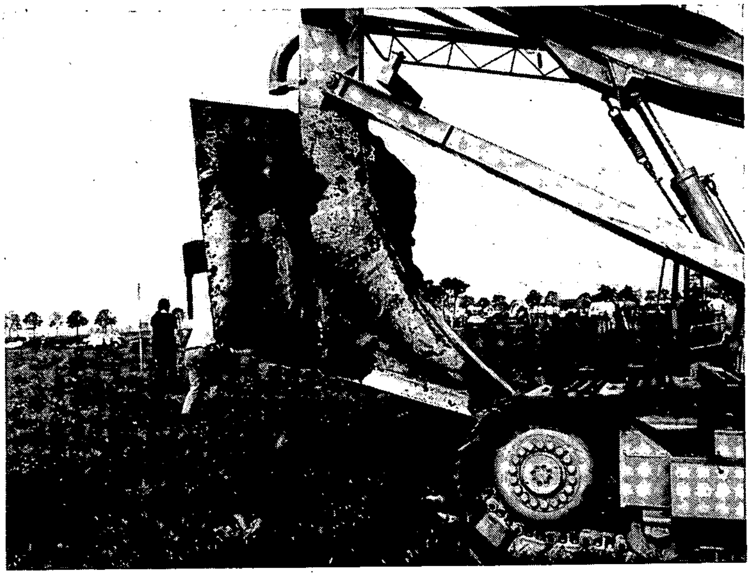


Fig.2. The ripperleg is positioned at an certain angle to the horizontal.The ripperleg is concave;the front plate extends over practically the whole depth

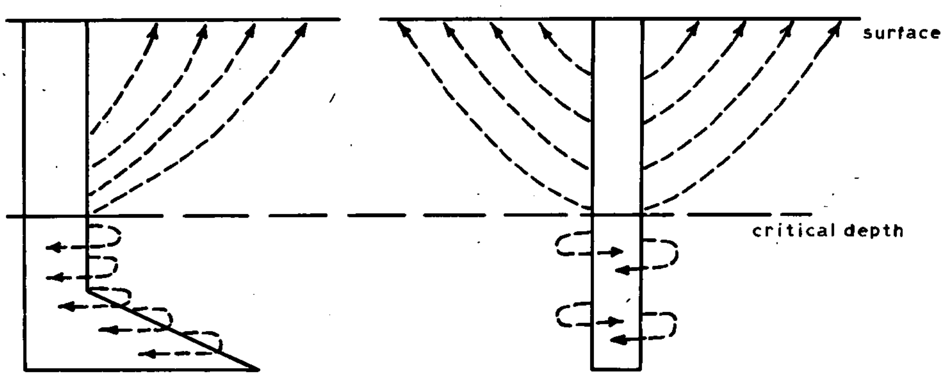


Fig.3. The ground is lifted down to a certain depth; below this level the earth slides along the ripperbody.

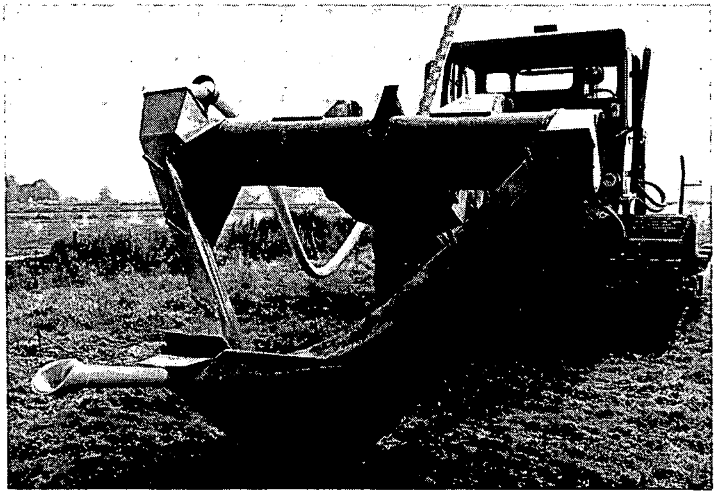


Fig.4. The "Willner" V-shaped plough body.

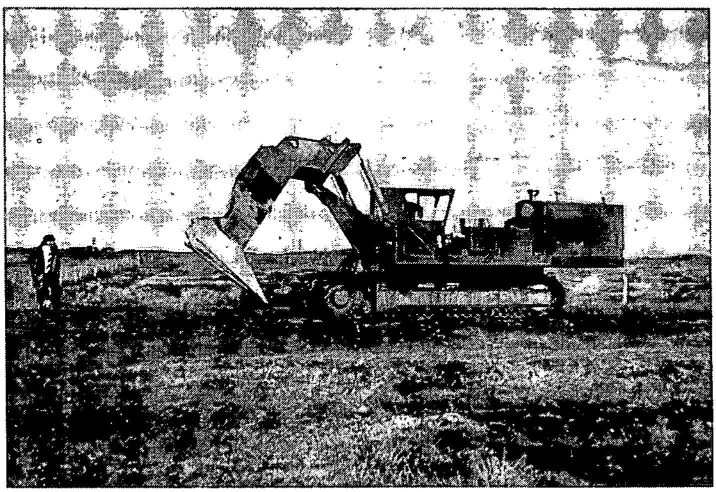


Fig.5. The Y-shaped plough body.

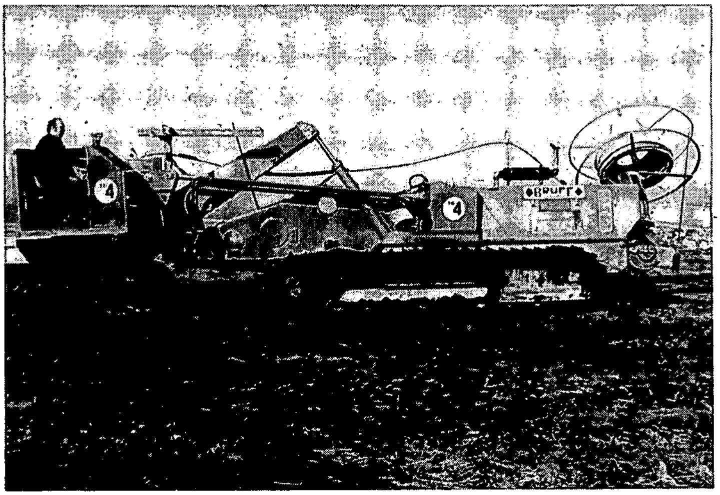


Fig.6. A winch is mounted at the front of the machine.

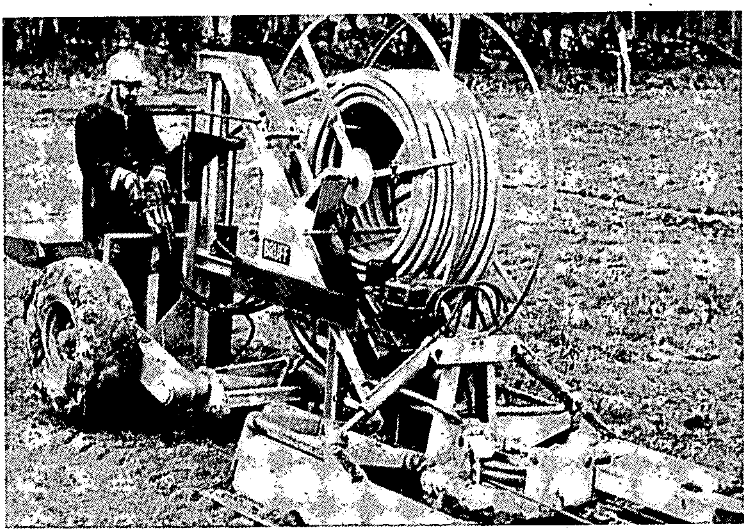
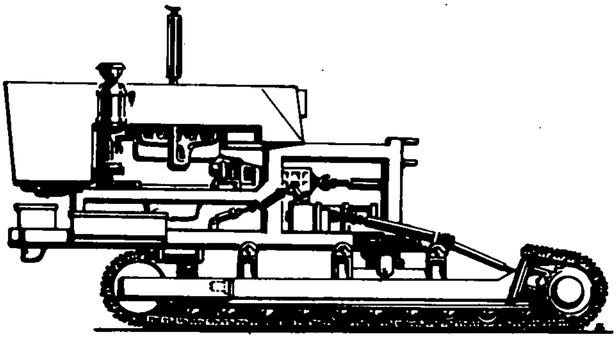
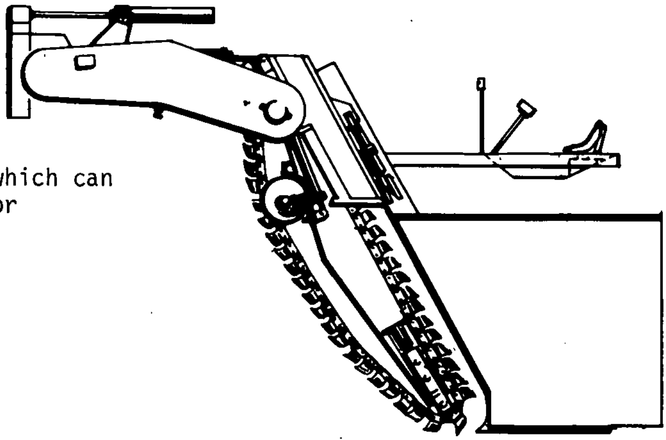


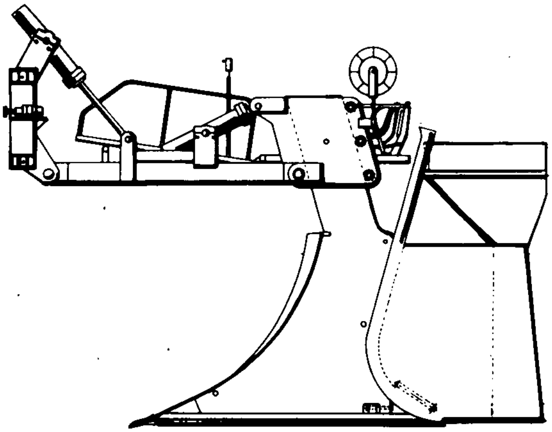
Fig.7. An unsophisticated machine (Bruff TG1) towed by a winch on a tractor. The cable and block in front are visible.



the tractor on which
the digging-chain or
plough body is mounted

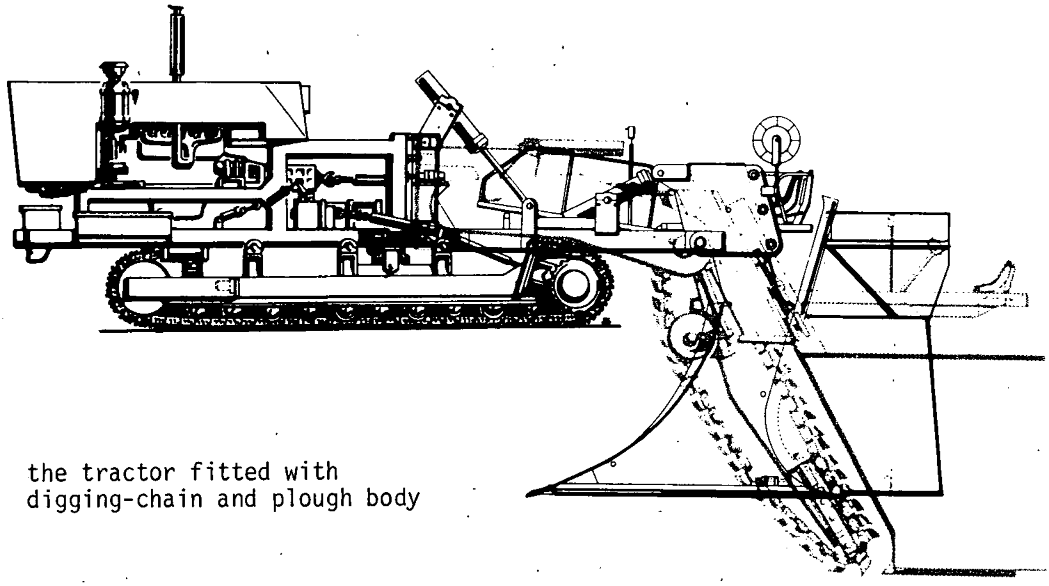


the digging-chain unit which can
be mounted on the tractor



the plough body which can
be mounted on the tractor

Fig.8. A trenching machine and trenchless machine combined.



the tractor fitted with
digging-chain and plough body

(Fig.8 cont.)

Paper 3.08

SOIL DISTURBANCE WITH DEEP WORKING TINED IMPLEMENTS IN FIELD DRAINAGE SITUATIONS

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Summary

Two types of soil disturbances may result at depth when using trenchless drainage ploughs, subsoilers and mole ploughs. The first, common at shallow depths, a loosening, fissuring disturbance ideal for the first two operations and secondly at great depths, a compressive, compacting disturbance suitable only for the latter operation. The working depth at which the transition between the two types of disturbance occurs, known as the critical depth, is dependent upon implement geometry and soil conditions. Increase in tine width and inclining the tine forwards increases the critical depth. Loosening the surface soil layers prior to deep tining increases the chances of loosening at depth. The use of shallow leading tines immediately ahead of the deep tine increases the critical depth without increasing the draught.

1. Introduction

Deep working rigid tined implements are in common use in drainage practice, examples being trenchless drainage machines, subsoilers and mole ploughs. The successful use of these implements is very dependent upon the way in which they disturb the soil. In trenchless drainage operations, the prime objective is to open up a slot to accept the drain pipe, without compacting or smearing the soil at pipe depth. Any such compaction would increase the water entry resistance and should therefore, be avoided. Successful avoidance depends upon loosening and fissuring the soil at depth. For effective subsoiling, loosening and fissuring are again required throughout the complete working depth. The requirements change when moling, where the objective is to form a stable compact channel at depth with loosened soil above.

The basic shape of implement used for all these operations is similar, being an inclined narrow tine, and yet the tine is required to deform the soil in different ways depending upon the need. This paper discusses the way in which tines disturb soil and highlights the major factors which influence the nature of the soil disturbance and the forces involved.

2. Soil disturbance with narrow tines

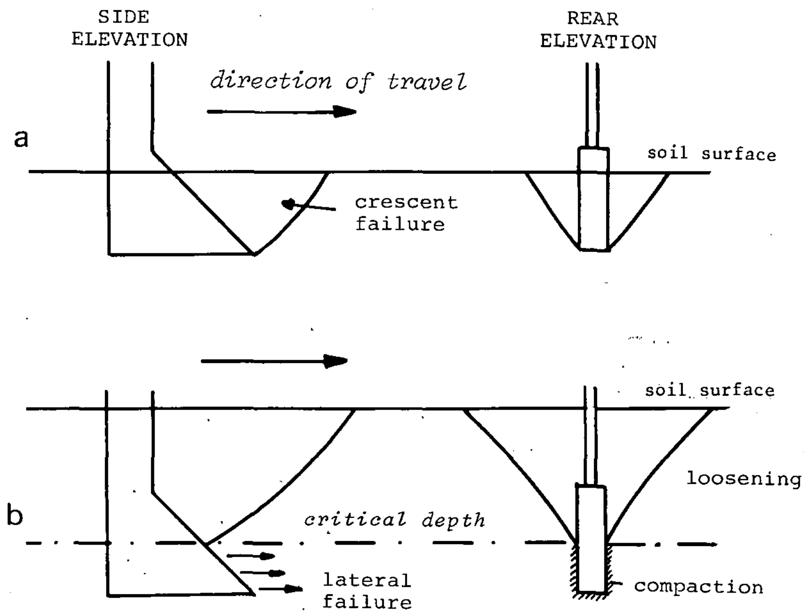

Fig. 1. Soil disturbance caused by trenchless drainage machine working at relatively shallow (a) and great (b) depths.

Two types of soil disturbance can occur with narrow tines, the type depending upon implement shape and soil conditions. This is illustrated in Fig.1 which shows the disturbance caused by a trenchless plough working at 2 different depths in a uniform soil. At the shallow depth, the soil is displaced forwards, sideways and upwards throughout the whole working depth.

This type of disturbance causes fissuring and loosening and is termed crescent failure. At the greater working depth the trenchless plough causes crescent failure in the upper soil layers, but at depth, the soil moves forward and sideways only (lateral failure) and this causes soil compaction around the lower part of the tine. This change in the type of soil disturbance with increasing working depth is found with all narrow tined implements, including subsoilers and mole ploughs (see Fig. 2). The depth at which the transition from crescent to lateral failure occurs is termed the critical depth and is specific for the particular tine and soil condition. This critical depth represents the maximum working depth of that tine for satisfactory soil fissuring and loosening at depth. Any further increase in working depth would cause compaction at the tine base.

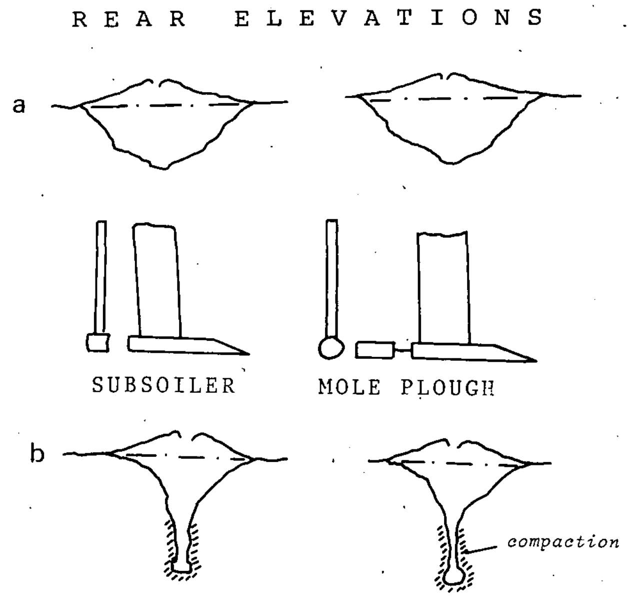

Fig. 2. Soil disturbance caused by a subsoiler and mole plough at relatively shallow (a) and great (b) depths.

To ensure minimum compaction and smear at depth, trenchless ploughs and subsoilers must always work above their critical depth. Conversely, the bullet on a mole plough must work below its critical depth to form the required compact channel. These drainage implements are required to operate at a given depth and whether their performance is satisfactory or not, will depend upon whether this depth is greater or less than their critical depth, under the prevailing conditions. An unsatisfactory performance can only be changed into a satisfactory one, if modifications can be made which will effectively change the position of the critical depth.

3. Factors influencing the position of the critical depth

The type of soil disturbance which occurs and hence the position of the critical depth, is dependent upon both the prevailing soil conditions and the implement geometry. The section of a narrow tined implement which controls the type of soil disturbance is the leading section at the working depth. There are two major implement factors which influence the critical depth position:

a) tine aspect ratio $= \dfrac{\text{working depth}}{\text{tine width}}$

b) tine inclination to the horizontal in the direction of travel (rake angle).

The smaller the aspect ratio and rake angle, the deeper the critical depth. Thus at a given working depth the wider the tine and the greater its forward inclination, the greater the probability of the critical depth being below the working depth.

The main soil factors influencing critical depth position are:

a) density

b) vertical confining stress resisting upward soil flow from depth (resisting crescent failure)

c) moisture content

d) texture and structure.

At any given moisture content. the lower the density and the higher the vertical confining stress, the shallower the critical depth. With constant density and confining stress values, increasing moisture content tends to decrease the critical depth.

Vertical confining stress can arise from forces both within the soil and on the surface. Wheels and tracks can impose confining stresses when positioned immediately above any failing soil. The internal confining stresses are dependent upon soil shear strength which tends to increase with decreasing moisture content. Severe surface drying, without a corresponding decrease in moisture content at depth, can result in the development of very high surface confining stresses which resist crescent failure, so effectively reducing the critical depth.

Table 1 shows for different rake angles and moisture conditions in a compact loam soil, the approximate aspect ratios at which lateral failure at depth becomes significant. It can be seen that crescent failure, over the whole working depth, continues to occur at higher aspect ratios with low rake angle tines than with high. As the surface layers become drier relative to the lower layers, wider tines at a given depth (smaller aspect ratios) are required to maintain crescent failure. Under soft wet plastic conditions the critical depth is near to the surface at almost all aspect ratios.

TABLE 1. Minimum tine aspect ratios where lateral failure at depth became significant on a compact loam soil

Mositure state	Tine rake angle	Aspect ratio
Friable throughout	90^{o}	6 - 8
	45^{o}	10 - 12
Hard dry surface layer, friable at depth	45^{o}	6 - 8
Friable surface layer, just plastic at depth	45^{o}	6 - 8

4. Implement design and field use aspects

Implement performance has been shown to be very dependent upon the position of the critical depth relative to the working depth of the implement. Every effort must therefore be made to get this relationship correct and the main factors that can be changed to achieve this are, tine width, tine rake angle and the surface confining stresses in the soil. A change in any of these factors, however, not only changes the critical depth position, it also influences the draught force. Wherever possible, the modification adopted should be the one which has the most desirable effect on the draught force, namely keeping it to a minimum. Results from soil tests in a compact sandy loam soil will be used to illustrate the effects of changes in tine width, rake angle and soil confining stress on draught and critical depth.

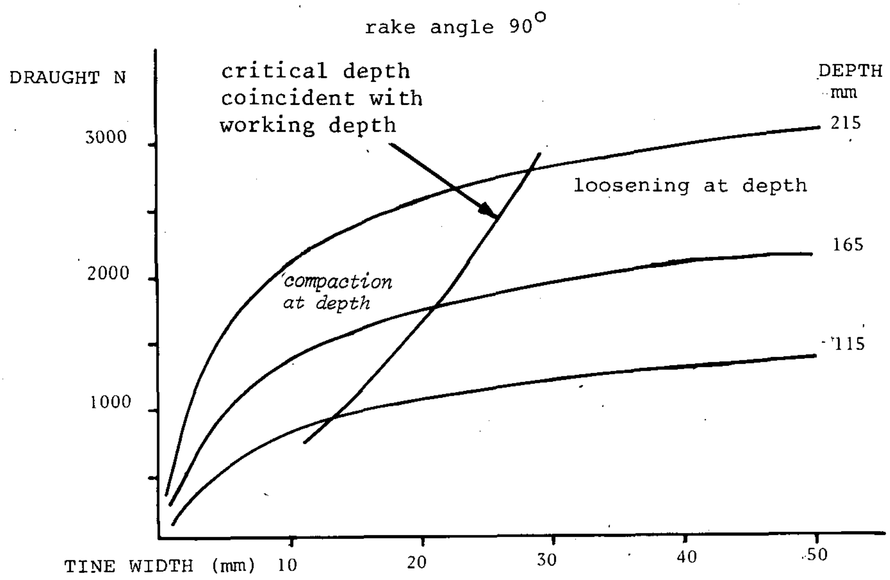

Fig. 3. Change in tine draught with tine width and depth.

Figure 3 illustrates for a vertical tine, how the draught changes with increasing tine width at different working depths. The tine width, where the critical depth is coincident with the working depth is also shown. It can be seen that the draught force increases with increasing tine width, but at a decreasing rate. The rate of increase is relatively small in the region where the critical depth is close to the working depth. Therefore, in situations where the critical depth of the tine is not too far above the working depth, increasing tine width will lower the critical depth to the required level, without too large an increase in draught. Tine width modifications can readily be carried out in the field.

The relationships between draught, tine width and rake angle are shown in Fig.4 together with the transition points where the critical depth is coincident with the working depth. Reducing the rake angle of a tine, at a constant working depth significantly reduces the draught and also increases the critical depth. There is little further to be gained in terms of draught reduction by reducing the rake angle much below 45^{o}. Changes in rake angle although not normally possible in the field, can be considered at the design stage.

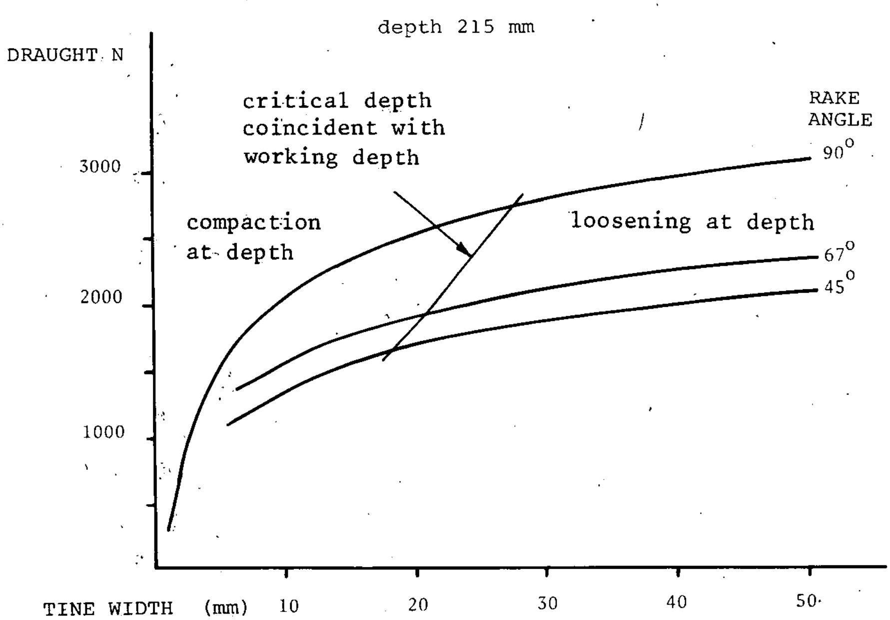

Fig.4. Change in tine draught with tine width and rake angle.

Surface confining stresses are very dependent upon moisture status and therefore, in certain circumstances, it may be necessary to allow the soil to dry or wet before a satisfactory job can be done. An alternative procedure is to reduce the confining stresses in the surface by loosening, before the deep tining operation is carried out. The loosening can be done as a separate operation or performed at the same time as the deep tining. In the latter case, this is best done by positioning shallow working tines immediately ahead of the deep tine, to loosen the surface layers in the region where soil failure planes would develop if the deep tine were used alone (see Fig.5). The leading shallow tines should be far enough ahead of the deep tine so that they do not restrict the upward flow of soil from the deep tine, i.e. at a distance approximately equal to 1 - 1.25 times the working depth of the deep tine. Relieving the confining stresses in this way increases the critical depth over a wide range of moisture conditions.

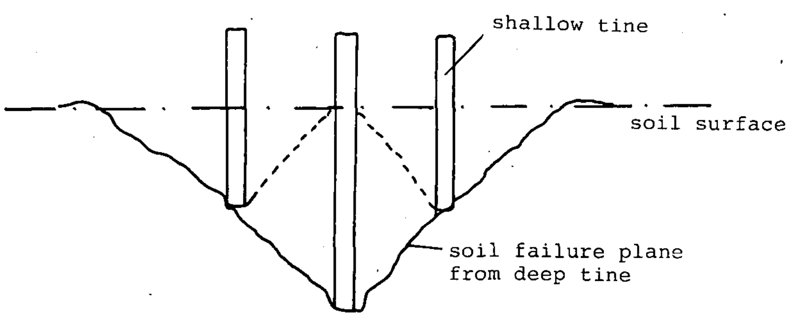

Fig.5. Position of shallow tines relative to soil failure planes created by deep tine alone.

Table 2 allows a comparison to be made between the forces acting on a single deep tine and those on a deep/shallow tine combination in a compact sandy loam soil. It can be seen that the addition of the shallow tines has had no effect on the total draught. The force on the deep tine is considerably less when it is preceded by the two shallow tines than when it is working alone. If the surface loosening and deep tining is therefore carried out as a 2-stage operation, opportunities arise to reduce the force input required from the power unit. In trenchless drainage operations, the shallow tining could be done as the machine runs back empty prior to the next working run.

When soil loosening is required, deep working tines should always be positioned far enough behind wheels and tracks, to prevent the latter effectively increasing the confining stresses on the soil being failed by the deep tine. Any surcharging effect would tend to reduce the critical depth.

TABLE 2. Draught forces of various tine combinations

Tine combination	Total draught N	Draught on deep tine alone N
Two shallow tines	300	
One deep tine	612	612
Two shallow tines + one deep tine	590	290

Tine rake angle	45^{o}
Working depth of shallow tines	70 mm
Working depth of deep tine	150 mm
Tine width	25 mm

5. Conclusions

a) Two types of soil failure can occur with deep working tines. At shallow depths a loosening, fissuring occurs (crescent failure) and at great depths, a compressive, compacting failure (lateral failure).

b) The working depth at which the transition between crescent and lateral failure occurs is known as the critical depth.

c) For minimum compaction in the region of the drain pipe in trenchless drain laying and for successful subsoiling, the tine must work above its critical depth. For successful mole draining the mole plough bullet should work below its critical depth.

d) The position of the critical depth is dependent upon tine geometry and soil conditions.

e) Increasing tine width increases the critical depth and increases draught.

f) Inclining the tine forwards to the direction of travel, increases the critical depth and reduces the draught.

g) Shallow leading tines ahead of the deep tine increases the critical depth without increasing the draught.

h) Loosening of the surface layers as a separate operation prior to deep tining reduces the total draught force compared with a single stage operation.

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Paper 3.09

TRENCHLESS DRAINAGE

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Summary

The development of drainage ploughs is discussed as well as the plough designs and performances.

Introduction

In much the same way that drainage materials industry was revolutionized by the development of corrugated plastic tubing in the last decade, the drainage industry is now being revolutionized by the drainage plough. Van Schilfgaarde (1971) predicted a swing towards plough installation of subsurface drains, primarily due to reduced installation costs. The move to ploughs has taken place in the 1970's at a much greater rate than even he envisioned. This "trenchless" method of drainage is now on the way to becoming a principle means of installing subsurface drains. High-speed installation and low maintenance costs combine to make profitability the major factor in forcing the drainage plough to its position of increasing importance in the installation industry.

Several important developments during the past ten years have made trenchless drainage practical. The first of these was the application of the concept of corrugations in the walls of plastic conduits for maximizing structural strength per unit weight of material. The second was the development of the laser automatic grade control system. Corrugating the tubing wall provided strong, light-weight drainage conduit with the longitudinal flexibility needed for coiling, for handling in long lengths, and the flexibility needed for bending during installation. Development of automatic laser grade control made it possible to install drains accurately to the proper depth and grade at the significantly higher speeds attainable by the plough. A third development was the adaptation of a prime mover capable of meeting the high draft requirements for trenchless drain installation. Such equipment was already available from within the heavy construction industry. A variety of both crawler and rubber tired vehicles are used with attachable ploughs. Self-propelled, single-unit ploughs are also manufactured that make use of available component parts from the heavy equipment industry,

Trenchless drainage concept

The plough-in method of drain installation has been termed "trenchless". In contrast, the trenching method involves both soil excavation and backfill operations. The "trenchless" method places the tubing at a prescribed depth in an open channel beneath a temporarily displaced wedge or column of soil. The plough blade is designed to lift and split the overburden soil as it moves forward. The lifting action causes a deformation and disruption of the soil upward and outward at an angle on both sides of the plough blade (Fig. 1). The tubing is fed in behind the plough blade before the soil falls back around the tubing. In some areas, the split soil mound, which is formed at the surface, is pushed back down into place by running the crawler track back over the soil mound. In other areas, this method is not used. In such cases, the surface soil is re-levelled by other means or allowed to resettle naturally.

The two major questions associated with the plough-in technique are:

1. Can the tubing be laid on grade at the desired depth within acceptable limits;
2. Does the condition of the soil after installation allow unrestricted flow of water into the drain?

Plough design and operational characteristics, along with soil conditions, determine to a great degree how well these two requirements are met. Fouss et al. (1971) demonstrated that, in a silt loam soil, grades can be held to less than ± 2 cm. However, the speed of installation and responsiveness of the grade control hydraulic system are key elements in determining how well grades are maintained. Some evidence has been obtained on reduced flow into ploughed-in drains, but the number of cases and the area involved is small compared to the large amount of tubing installed.

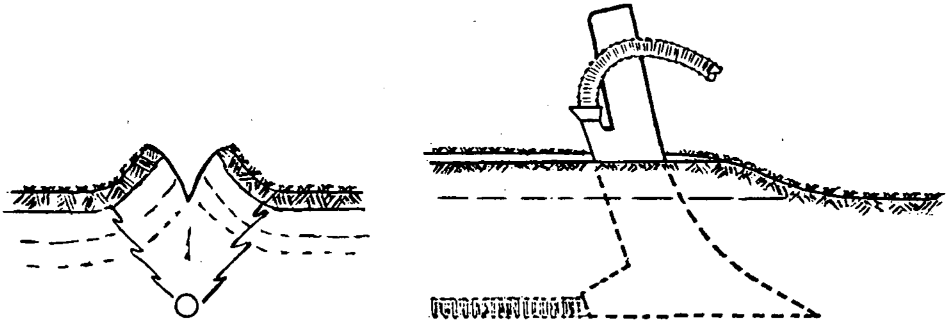


Fig.1. "Trenchless" installation of subsurface drains. Lifting action disrupts and shatters soil at an angle outward and upward.

Drainage ploughs

There are now more than 15 manufacturers and distributors of drainage ploughs in the United States and Canada (Darbishire, 1977). Latest estimates place the number of ploughs in operation in North America at over 200. With the coming of each drainage season (spring and fall), the number of drainage ploughs increases. Often, the new plough contractor is a former trenching contractor who was not able to successfully compete with the ploughs in his area and was, thus, forced to join the ranks of "trenchless" drainage contractors. Speed of installation and low maintenance costs are the two factors mainly responsible for the change from trenching to plough-in drainage.

Early in the development period of "trenchless" drainage, the ploughs were used to install smaller diameter lateral lines and trenchers were used to place the main drain collectors. Currently, ploughs are being used to install both laterals and mains. Corrugated plastic tubing up to 10 inches I.D. is routinely ploughed-in to depths of six feet. In north-western Iowa and southern Michigan, plough installation of 12 inch tubing is anticipated for this coming spring (1978). Concrete mains as large as 16 inches I.D. have been installed by the "trenchless" method in Iowa.

Drainage ploughs consistently out-perform trenchers in rate of installation by a factor up to 10 to 1. For a ten-hour day, the installation of up to 50,000 feet of drain tubing per day has been reported. Installation of 20,000 feet per day on a routine basis is not uncommon. This compares to 5,000 or 6,000 feet per day for conventional trenchers.

Development of drainage ploughs

Serious use of ploughs for installing agricultural drains began in the 1960's. In the United States, the initial research was on the adaptation for lining a mole drain channel with a smooth-wall plastic liner (Fouss, et al., 1964). Equipment was developed that would form a cylindrical conduit from a flat roll of 15 mil. plastic (PVC), would interlock (zipper) the seam, and would place it in a mole channel formed with a modified mole plough. Results showed these liners held up for a period of up to four or six years, but failure by collapse eventually occurred. Although the flow capacity was severely reduced by this failure, the reduced channel still conducted water. These collapsed lines have continued to flow and to provided water table control on one test site for the 15-year period since installation.

Later research by the United States Agricultural Research Service (Fouss, et al., 1971) resulted in the construction and testing of a dual, long-beam "floating" type plough, together with the development of an automatic laser grade control system. Field testing of this prototype plough and grade control equipment demonstrated the feasibility of high-speed plough-in of corrugated plastic tubing to depths of up to 5 1/2 feet.

During this period, drainage plough technology advanced considerably. Several European ploughs were introduced into North America. These form the basis for most of the ploughs on the market in the United States and Canada today.

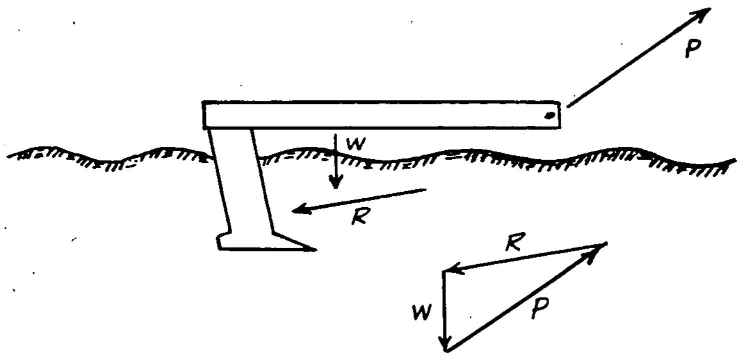


Fig. 2. *The long-beam floating type plow seems to "float" because of the balance of forces on the plow.*

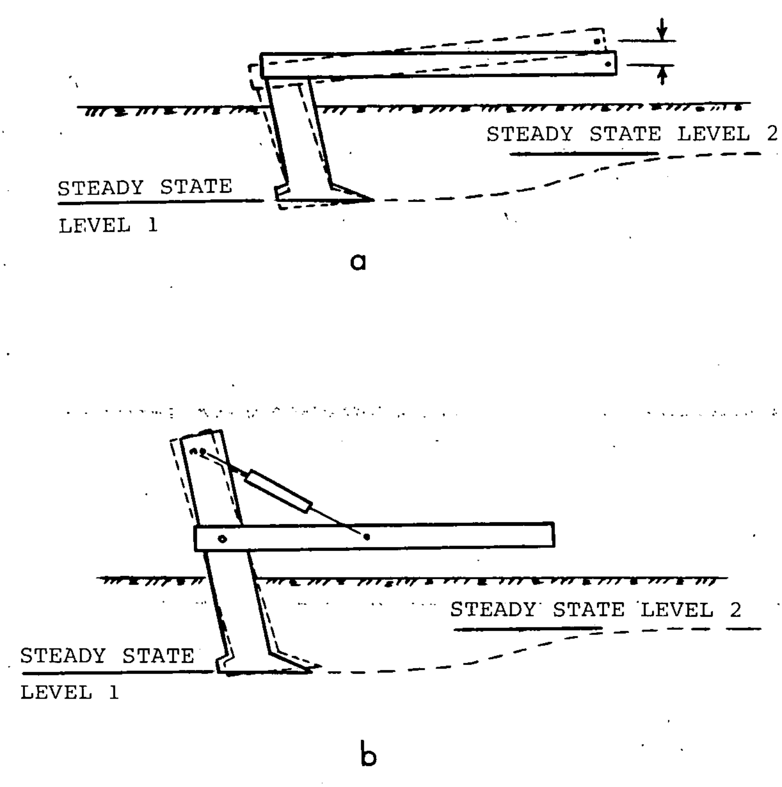


Fig. 3. *Plowing attitude and, hence, plowing depth is controlled by:*
a) raising or lowering plow-hitch point; or
b) changing blade angle.

"Floating" plough principle

The term "floating" type plough comes from the fact that, as the plough is pulled through the soil, the soil drag forces on the blade are in balance with the tractor draft and plough gravity forces so that the plough seems to "float" through the soil (Fig.2).

Ede (1961) conducted field trials on the "floating" beam plough in England. The "floating" plough has since become the standard in the industry. Ploughing depth is controlled by changing the attitude or angle of the plough This can be accomplished by either raising or lowering the hitch point (Fig. 3a) or by rotating the plough blade (Fig.3b).

Fouss et al. (1971) showed that changes in ploughing depth, in response to hitch height changes, were approximately linear, but not directly proportional. For example, his results showed that, for a change from one steady state flow position to another, a 4 cm vertical displacement of the hitch resulted in a 5 cm change in ploughing depth.

This type of a response is characteristic of all floating type ploughs. It is governed somewhat by plough blade design, but is influenced more by the soil drag forces on the blade as plough depth changes. These drag forces vary as some mathematical power of the depth. Draft curves for ploughs, developed by Jackson (1977) for clay, sand and silt are shown in Fig.4.

The plough is also subject to change in depth as a result of changes in texture or soil consistency along the plough path, even though no change has been made in hitch-point elevation. Because of these variations in drag forces on the plough, grade control requires more than keeping the hitch-point height on a line which is parallel to the desired drain gradient. Information on both the elevation of the plough point and plough attitude is needed for precise control. Ideally, two detectors would provide the needed feedback information; however, both attitude and elevation can be monitored by proper positioning of one detector unit.

Most control systems in use on drainage ploughs today utilize only one detector for feedback and are of the on/off type. Fouss (1971) treated the subject of control systems and positioning of single detector units in detail

For an on/off control system with one detector unit, he recommended a detector position of 0.833 times the long-beam length towards the rear of the plough from the hitch point.

The hydraulic system for operating the controls must be responsive and have a minimum time lag.

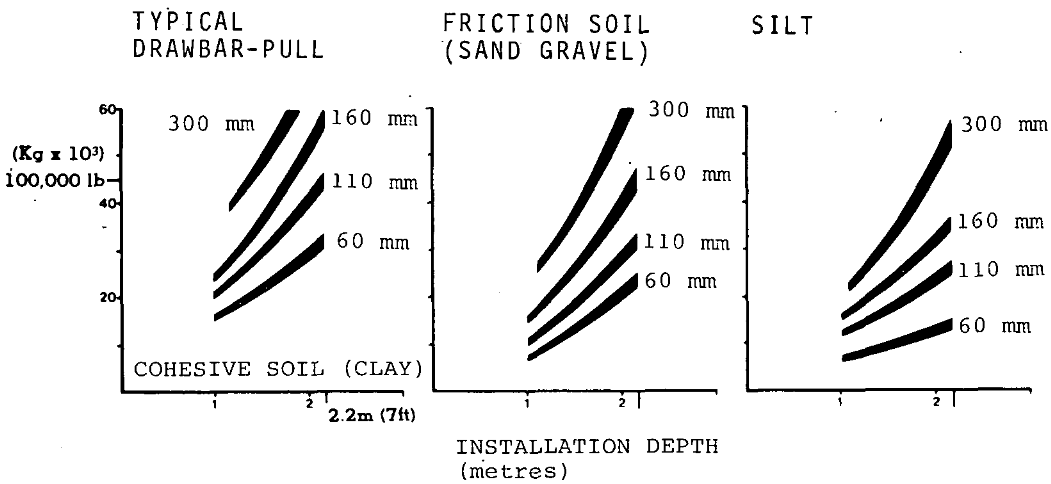


Fig. 4. Drawbar pull required as a function of installation depth for various size drain tubes in different kinds of soil (Jackson, R.T., 1977).

Plough design

The "floating" principle is incorporated into the design of most drainage ploughs. However, the depth-gage wheel type plough can be used in areas where land slope is uniform and the ground surface is relatively smooth.

Such a plough has been used extensively in the Imperial Valley, California, as shown in Fig.5 (Willardson, 1970).

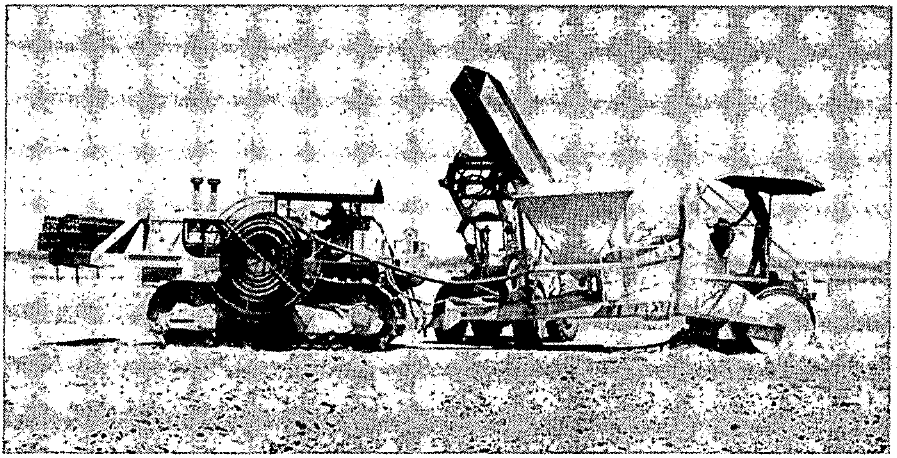


Fig.5. Depth gage wheel type plough, Imperial Valley, California, equipped to place gravel pack around drain tubing.

Floating type drainage ploughs

There are at least five different types of ploughs produced or distributed in the United States and Canada that utilize the "floating" plough principle. Based primarily on the type of linkage and hitch-pount location, drain tube ploughs can be categorized as follows:

I. Actual Hitch Point

- A. *Movable*
- B. *Fixed*

II. Imaginary or Virtual Hitch Point

- A. *Double Roller*
- B. *Double Link System*
- C. *Tiltable Parallel Link System*

The linkages of these plough types are illustrated in Fig.6.

The hitch points, real or imaginary, for all but the fixed-hitch-point plough are located near the front end of the tractor. This procedure brings the resultant force on the prime mover near the centre line of the tracks at the ground surface for the purpose of providing uniform load distribution

ACTUAL HITCH POINT

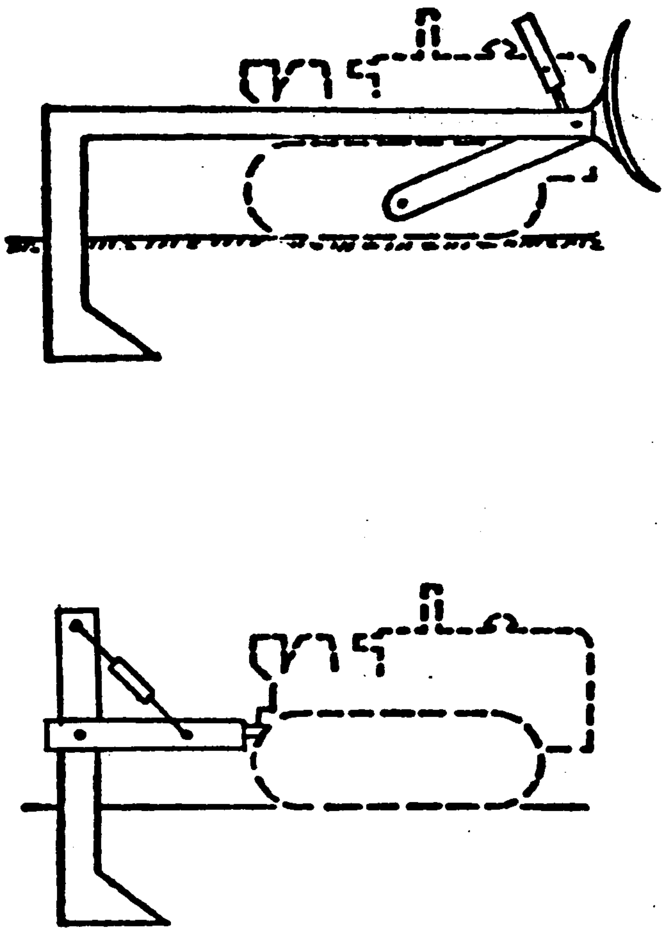
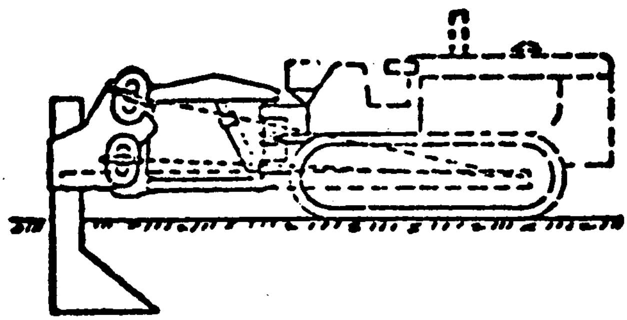
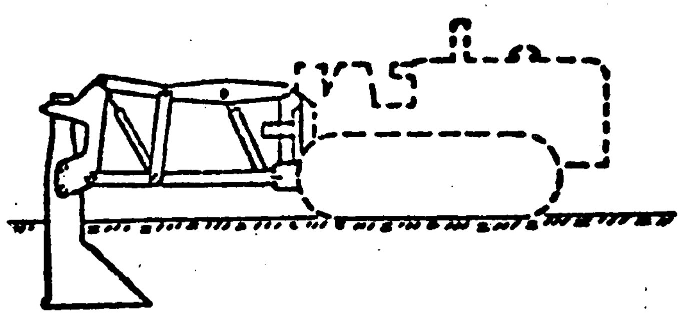


Fig. 6. Plow types based on hitch-point linkage.

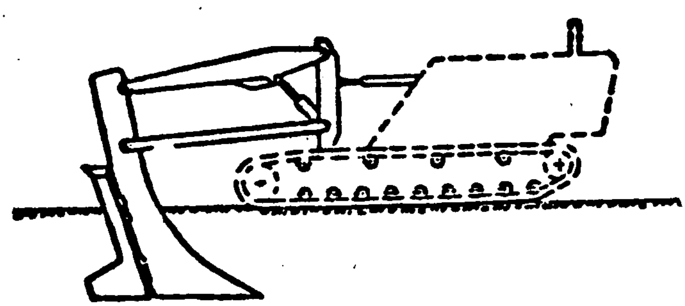
IMAGINARY OR VIRTUAL HITCH POINT



double roller



double link



tiltable
parallel link

(Fig.6 cont.)

on the tracks. The track load is increased by as much as 30 per cent of the normal tractor weight by the vertical component of the plough draft force. Optimum traction is attained by maintaining the track pressure uniformly under the entire track surface (Fig.7). The forward hitch-point position and free-floating action also minimizes the backward tipping moment on the prime mover, which is a major problem for fixed-plough mountings or where ploughs are attached to the draw bar at the rear of the tractor.

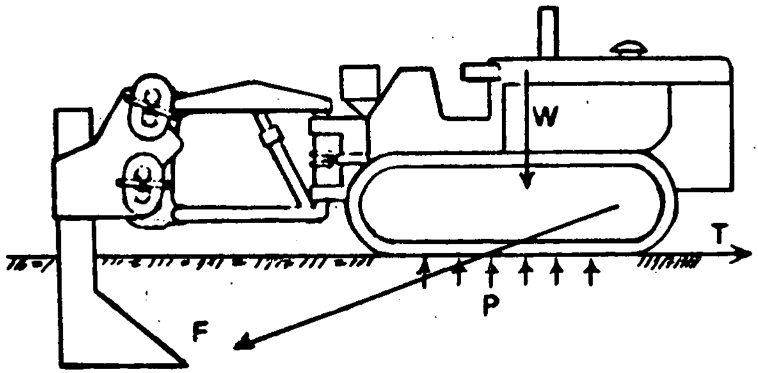


Fig.7. Uniform track pressure increases traction. The long-beam "floating" design is used to increase track load and give uniform load distribution.

The moveable hitch-point plough is shown in Fig.8. The configuration used here utilizes a twin beam with each of the two beams attached to the rear of the dozer blade. The attitude of the plough blade with respect to the beam is manually adjustable by using the two turn-buckles shown at the rear of the plough. Grade control is achieved by moving the hitch point up or down with the dozer blade hydraulic system. The hydraulic cyclinder at the rear of the crawler is for lifting the plough out of the ground only. During the plough-in mode of the plough, this cylinder is in the float mode. The hitch point is a "pin" connection; thus, the plough is mostly free of pitching action of the crawler tractor.

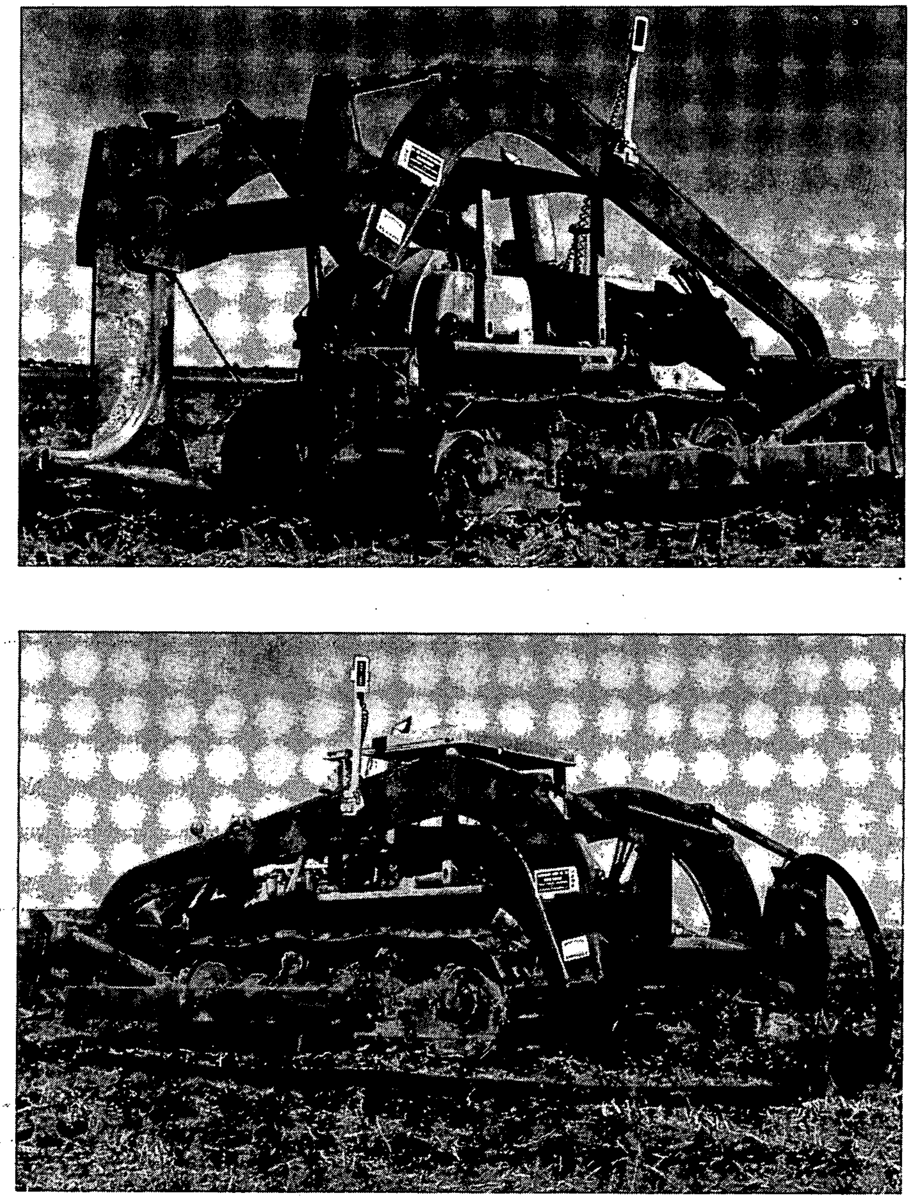


Fig. 8. Movable hitch point drain tube plow. This long, twin-beam floating type plow has its hitch point on the rear of the dozer blade (FOUSS et al., 1971).

The fixed-hitch-point plough utilizes a shorter beam than the other ploughs but achieves attitude control by changing the plough blade angle; the hitch remains fixed. The Hoes plough is an example of this plough type.

The concept of a roller-type floating plough with a virtual or imaginary hitch point was investigated by Ede (1961, 1965). The Badger plough, manufactured in England and distributed in Canada, makes use of this principle. The blade and tractor are connected through a pair of rollers which run on a curved, vertical track which is mounted on the rear of the tractor. The centre of curvature of this roller track, located near the front of the tractor, is the virtual hitch point.

The double-link plough utilizes two non-parallel links that provide depth control, combined with free-floating action. It is also a long-beam floating type plough with an imaginary or virtual hitch point. The rear link simulates plough rotation about the virtual hitch point and can be operated in a free-floating mode or with controlled constraint for attaining the desired load transfer to the tracks. The Zor and Krac ploughs are examples of this type of plough.

The parallel link system, with a forward-backward tiltable mounting, provides both plough depth and attitude control. The Barth and ADS ploughs are examples of this type of plough.

The pitching action of the tractor over uneven ground is a major problem for all trenchless methods. The problem is minimized, however, where the free-floating action of the plough is incorporated in the linkage design. The plough blade is thus nearly isolated from most of the pitching movements of tractor.

Plough performance

As mentioned in an earlier section, the standard of plough performance is based on how well the tubing can be installed to grade and still leave the soil in a condition that allows water to move freely into the drain. Satisfactory results, based on these requirements, depend on a number of factors, among which soil conditions, plough responsiveness, plough speed, and operator skill are of prime importance.

Soils vary from light sandy, to loam, to heavy clay, and from very dry to very wet. Prime-mover traction and soil compaction are more likely to be a problem in wet or high-clay soils. The occurrence of rocks constitute a serious problem in drain installation. It has been observed that ploughs perform better in rocky soils than do most trenchers. In areas where rocks are a problem, locations where rocks are encountered are flagged at the time of contact by the operator for later inspection and correction by the crew, if needed. Depending somewhat upon the hitch arrangement, a plough will tend to either push the smaller rocks aside or, where larger rocks are encountered, the plough will move sideways around the rock. If the plough is deflected upward a small distance by a rock, where slopes permit, an alert operator will make a small adjustment in grade from the point on and, thereby, not leave a hump in the line for later correction. Rocks as large as 100 cm in diameter have been lifted completely out of the ground as the plough passed through the soil.

It is well-known that mole drain ploughs compact soils. By association, there is a tendency to expect similar compaction with drain-tube ploughs. It is true there is a similarity between mole ploughs and drain-tube ploughs; but because of their very different blade designs, the compactive action on the soil is entirely different. The mole plough is designed to form a channel in the soil by radial compaction so that the mole drain will hold its shape for a long period of time. Consequently, the compaction of the soil outward from a central hole by a bullet-shaped plug is desirable and is a part of the design for moles. However, the drain-tube plough is designed to lift and disrupt the soil with as much disruption and as little radial compaction as possible.

Spoor (1976) described a model for soil disturbance and the accompanying compaction for sub-soilers. For a flat blade at a constant rake angle, his model shows loosened soil above some critical depth and compacted soil due to sideways soil movement below that depth. The model results were substantiated by laboratory tests.

Data are not available for compaction by blades with compound leading edge slopes. The sub-soiler model does not seem adequate for describing the effects of drainage plough blades. Drainage plough blades are designed to impart an upward movement to the soil from the blade cutting tip for some distance back and upward, thus minimizing compaction and creating a large amount of disruption to the soil as it is broken loose and heaved upward.

Irwin (1971) reported on measurements of soil density around ploughed-in drains. He found minimal compaction under and to the sides of the drain (<5 per cent). The unit weight of the soil was reduced by more than 15 per cent in the disturbed soil zone. The disturbed zone extended upward at an angle of about 45 degrees on either side of the drain.

Field investigations of trenchless drain installations were conducted in several different localities in the United States in 1977, under different soil conditions, to evaluate the immediate effects of ploughing around the tubing. In Kentucky, tubing was ploughed-in in a clay-loam soil at a depth of approximately five feet where the soil was extremely dry and in the same soil at another place where it was extremely wet. In the dry, clay-loam soil, the shape of the plough cut was precisely outlined to the exact shape of the plough point, with extreme fracturing and loosening of the soil above and outward at an angle of about 30 degrees from the vertical on both sides. In the wet soil, the cut was not greatly different. The fracturing was less evident, but the soil disruption was equally great. There was no measurable compaction under or around the tubing in either case.

In Iowa, at three different locations in a loose, well-aggregated black soil with a heavy light-coloured clay subsoil, the ploughed-in tubing became covered for a full 180 degrees over the top with 8 to 10 inches of loose, aggregated black soil. In this case, the well-aggregated and loose surface soil flowed freely down behind the plough blade and tubing boot. The drain was, thus, effectively blinded by a covering of highly-permeable, well-aggregated soil. Penetrometer measurements around the base and on each side of the tubing showed no measurable differences due to compaction.

The natural blinding action of the plough was observed to occur also in a sandy soil near Tifton, Georgia. The surface layer was a loose, dry sand which flowed down freely behind the tubing boot and covered the top of the tubing. This is desirable where the surface material is highly permeable and stable, but it could cause siltation problems if the surface material is extremely silty or is very fine and uniform or if a filter is not used.

With a filter, a covering of medium to coarse sand around the tubing could enhance inflow.

High-speed installation

The high speed of the trenchless drain plough has brought about a large increase in rate of installation of agricultural farm drainage. An example of the effect of the plough is the operation of drainage contractors, G. and R. Lazure, Quebec, Canada. Changing from a business built entirely on trenching in 1968 to an essentially full-plough operation in 1977, their annual installations increased from slightly less than 1 million feet to more than 15 million feet per year.

Contributing to their high production level is a well-managed organization where crew training and unique plough designs virtually eliminate lost time. The 15 million feet of tubing installed last year was accomplished in 32 work weeks with 8 ploughs operated by 6-man crews working 50 hours per week.

Their ploughs, which are of the double free-floating link type, are mounted on D8H crawler tractors. The basic plough blade is for 4-inch (or 3-inch) tubing; but has a removable attachment that can be added or taken off in less than three minutes in order to convert the plough blade to one that can install 6-inch or 8-inch tubing. Thus, they plough-in both mains and laterals. For the mains, an initial pass is usually made with the plough at about one-half the final depth. Following this initial cut, the tubing is ploughed-in to depths up to 7 feet with a D7F in tandem with the D8H. Their target for 1978 is 20 million feet of installed drains.

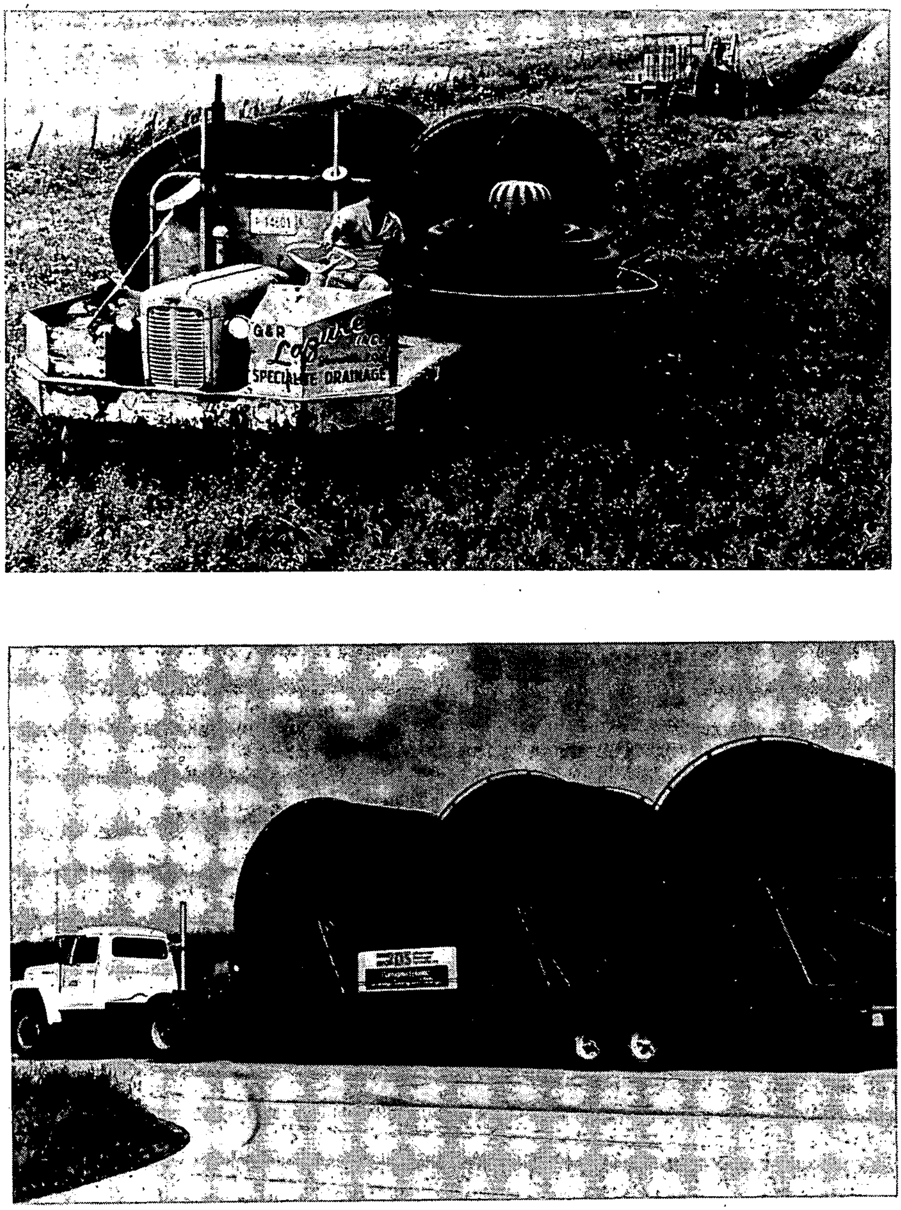


Fig.9. Farm wagon built for stringing tubing from regular 250 foot rolls and big three-spool trailer with capacity of 15,000 feet.

Stringing tubing

Because of the high plough speeds, stringing of the tubing in the field prior to installation is a major item. To reduce both labour and material costs and to meet the high-speed stringing requirements of the plough jumbosized coils which are mounted on big-spool trailers have been developed Fig. 9 shows a stringing wagon which is commonly used for handling the regular 250 foot tubing coils and the big three-spool trailer. Each spool, loaded at the factory, carries up to 5,000 feet of 4-inch tubing, making the total capacity of the unit 15,000 feet. In addition to the reduction in stringing time, more than 50 connections per load are eliminated, thus saving additional time and materials.

The future of ploughs

In a relatively short time, the trenchless method of drainage has come to the forefront as an effective and economical method of installing drains. Drain plough designers and engineers have done an excellent job in meeting the requirements of a new drainage method. Contractors are finding that maintenance costs are low, and the rate of installation gives a competitive edge that is shifting the industry from trenching to trenchless drainage. Whereas, the drain tube plough started from a zero base a few short years ago, it is not difficult to imagine that the trenchless method will dominate the industry within the coming decade.

The rapid change envisioned is fraught with the possible danger of a decline in the quality of agricultural drainage. Shifting to a higher rate of installation requires improved procedures for maintaining accuracy of placement, both for drain grade and depth. While the concurrent gain in control technology may be adequate to ensure the required accuracy of installation, the means for ensuring acceptable performance in the field has not kept pace. Developing procedures for maintaining quality drain installations is a major challenge for drainage of the future.

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Paper 3.10

DRAIN LAYING QUALITY SURVEY

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Summary

The efficiency of field drainage demands particularly careful work. A survey of the laying quality has been recently held in France. The present paper gives a quick description of data handling and processing and then analyses the respective influence of various parameters, among which those relative to soil and machines appeared to be most important.

1. Introduction

The recent progress of field drainage in France can be explained first by a generalized mechanization and also by a more extensive information about it.

The mechanization development is linked with the cost lowering in constant francs, this lowering being itself correlated with the increase of laying speed.

One could then expect the laying quality to decrease if a certain number of recommendations or norms were not given to works consultants and applicants.

Because of this, the Ministry of Agriculture has edited a book of recommendations (Cahier des prescriptions communes, 1971) for drain works, referred to as "CPC" in this paper. In this document, among other rules, several tolerances were fixed, according to the laying technique (handmade, by excavator, by trencher, by trenchless machine).

In 1973 and 1974, the CTGREF (Ministry of Agriculture, Technical Center) held a quality survey to estimate the respect of CPC recommendations.

It was found that too severe tolerances could not be respected without considerable increase of costs.

However, the results were alarming: a high number of negative slopes and laying errors did appear.

In 1976, presuming that no quality increase had occurred, and searching for more recent and objective datas, the CTGREF held a new quality survey in whole France, which gave data for a more refined analysis.

This paper briefly describes the measurements, presents the data analysis and summarizes the conclusion of it.

2. Effects of a bad laying of drains and collectors

Effects of a negative slope

A negative slope is schematized on Fig. 1.

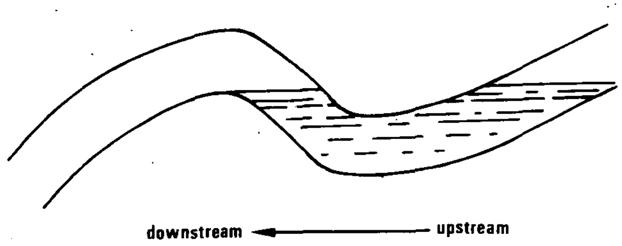

Fig.1. Negative slope of a drain.

It is clear that such a situation enables a stagnation of water at the end of a drainage period; this area, when the soil is drying, attracts crop roots and, thus, clogging by roots.

In addition, the slope change lowers the mean water velocity and provides sedimentation of solids transported (loam, fine sand, roots), and thus inner clogging.

Finally, the flow under pressure is a factor of drain instability.

Effects of a laying error

A laying error provides an increase or a decrease of the drain slope. In the second case, the effects are similar to those of a negative slope, though less important.

Case of the collectors

Particular care must be observed for collector laying, these pipes being the major organs of the drainage layout. Thus, collectors must formally present neither negative slope nor laying error.

3. Quality survey

The quality survey consisted in checking the profiles of several drains on each site. In order to obtain homogeneity among different sites, it has been decided to check about 200 metres on each site, with one point per metre. In order to check the depth of pipes, the soil surface elevation was measured every fifth metre.

In view of avoiding the risk of errors, the pipe was made visible before measurement. In the case of trenches, checking was made just after laying. In the case of trenchless machines, it was necessary to extract the backfill with an excavator. The levelling was made with a level combined tacheometer. The sighting pole had a mobile shoe (Fig.2).

For each drain, several informations were handled, among which the type of machine, the control system, some soil characteristics (texture, rock content, water content, stability, terrain aspect), and the machine velocity. These data were recorded in an explicative data file.

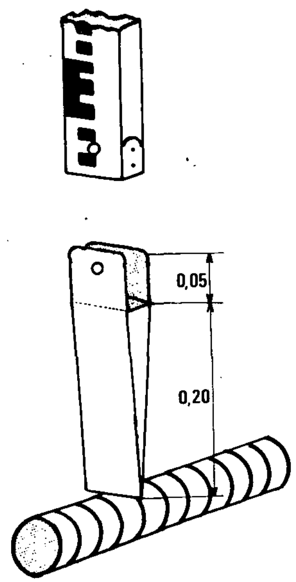

Fig.2. Pole and its shoe.

4. Data analysis

The analysis involved the three following steps:

- pipe slope draft;
- creation of an explicated data file, from the descriptive parameters of laying quality;
- variance analysis of this file, combined with the explicative data file.

The two last steps will be developed:

- the second one is in fact an automatic notation of the drain laying quality. The criteria were relative to underlayings, negative slopes and laying errors.

The underlayings, i.e. the points where the drain bottoms was less than 0.80 m deep (according to the trench drainage CPC), could be detected every five meters; the total number of such points was counted.

A negative slope was counted any time a point was more than 3 mm below the immediate downstream point. The length of the negative slope was estimated as the distance between the highest downstream point and the first upstream point at the same level. The amplitude of the negative slope was estimated as the difference between this level and the level of the lowest point in the considered area.

The negative slopes were classified into seven classes, according to the importance of their amplitude. One counted for each drain the number of negative slopes of each class, and their total number. A special count in terms of presence or absence of negative slope, was made for the first five meters of each drain, because of their importance and the hand-laying of this part.

The evaluation of laying errors implied existence of a reference pipe slope design. In the absence of such a draft for most projects, it was decided to estimate it practically and theoretically, from the level data. A least-squares method was used to adjust the levelling to an optimal line having at most one slope change. The theoretical design was thus composed of two half straight lines, the slope of which was necessarily positive.

The laying errors were then detected and classified in five classes determined from a tolerance depending on the mean slope (according to the trench drainage CPC). For each drain, the number of laying errors of each type was counted.

- The third step analysed the data file for each drain, involving explicative data from field observations and mean slopes, and explicated data obtained at Step 2.

The treatment was a variance analysis, executed for each couple explicative datum - explicated datum. When the sample size was large enough, additional treatments were performed for files reduced to a given value of an explicative variable, for testing the influence of a given parameter, with respect to another.

The general results of this analysis are listed and commented thereafter.

RESULTS OF THE VARIANCE ANALYSIS

EXPLICATED DATA \ EXPLICATIVE DATA		MACHINE	TRACTION	RIPPER	CONTROL SYSTEM	SERVOING	WETNESS	STABILITY	TEXTURE	STONE CONTENT	SPEED	TOPOGRAPHY	SLOPE
number of classes		5	3	2	6	2	6	3	5	5	5	4	4
NEGATIVE SLOPES WITHIN FIRST 5 METERS	0												
	1												
	2												
NUMBER OF NEGATIVE SLOPES	NC	-		-	I		I	-	I	I	I	I	I
LENGTH OF NEGATIVE SLOPES	LC	-		-	-		I	-	I	I	I	I	I
AMPLITUDE OF NEGATIVE SLOPES	0	-		-	I		I	-	I	-	I	I	I
	1	-		-	-		-	-	I	I	I	-	I
	2	-		-	-		I	-	I	I	I	I	I
	3	-		-	-		I	-	I	I	I	I	I
	4	-		-	-		I	-	I	-	I	-	I
	5	-		-	-		I	-	I	-	-	-	-
	6	5		-	-		-	-	-	-	-	5	I
LAYING ERRORS	0	-		-	I		I	-	I	-	I	I	I
	1	-		-	I		I	-	-	-	I	I	I
	2	-		-	I		-	-	-	-	I	I	I
	3	-		-	-		-	-	I	-	-	I	-
	4	-		-	-		5	-	-	5	-	-	5

LEGEND

- \- non significant test
- 5 significant test
- I highly significant test

5. Results

The results of the 1976 survey (9500 levelled points) are not much different from those of the 1973-74 survey (3850 levelled points) as well for the negative slopes as for the laying errors. The main difference lies in the fact that the first survey involved few sites held with use of trenchless machines and of laser control systems.

The first analysis of the 1976 survey results shows that the main problems are:

- negative slopes at the beginning of drains (downstream, which means near the junction with the collector)
- laying depth often too small
- high number of negative slopes
- high number of laying errors.

The variance analysis (see above) allows to precise what parameters have major influence.

Influence of machines and control systems

The results do not provide much difference between the two types of machine, as well as for the negative slopes as for the laying errors.

The control system by poles or rotating laser give a similar quality which appears better than that obtained by other systems.

It seems that the linear laser system provides negative slopes, which is paradoxal. The use of it is probably at the origin of these results.

The radio control system provides the highest number of laying errors.

Influence of machine velocity

For trenchers, the increase of velocity results in a decrease of quality. The phenomenon is less apparent for trenchless machines, but there was lack of high speed machines during the survey.

Influence of soil

The more the slope is important, the smaller the number of negative slopes; however, the number of laying errors increases with the slope. The operator is probably less careful in this case.

The soil moisture has also an influence: the number of negative slopes is almost the same in wet and in dry conditions; but the number of laying errors increases with wetness.

It is not possible to conclude about the part played by the stone content, because of the small size of the sample and of the particularly dry period in 1976.

About texture, the best laying is obtained in loamy soils, followed by clay soils and then sandy soils. The number of negative slopes is more important in sandy soils (instability of trench bottom; generally flat topography, high speed of machines due to a smaller resistance).

Topography

It is paradoxal that the number of negative slopes is higher in regular topography. This result can be explained only by a more careful work in difficult conditions.

As a conclusion, it appears that the drain laying quality, especially characterized by the number of negative slopes and laying errors and by the laying depth, is not mainly influenced by the type of machine and control system, but by the physical soil conditions (slope, moisture, texture, stone content, topography) and by human parameters depending on the draining staff and first of all the machine driver. The results given in the present paper show that the survey of drain laying quality must be now a major task for drainage consultants and applicants, but also that contractors must do an important effort of staff training, especially to make them understand the consequences of their work. Machine constructors have also to play a part by designing equipment able to make the driver's task easier. Finally, it would be emphasized that the objective character of the survey and of the data analysis gives conclusions for the french conditions only.

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CAHIER DES PRESCRIPTIONS COMMUNES pour la fourniture et la pose de tuyaux de drainage et ouvrages annexes. Min.Agric.France (1971).

Paper 3.11

A METHOD TO PREDICT CHANGES IN HYDRAULIC CONDUCTIVITY CAUSED BY THE TRENCHLESS AND TRADITIONAL DRAIN PIPE LAYING TECHNIQUE

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This paper will be published in the journal Agricultural Water Management and therefore only the summary is presented here.

Summary

A method was developed to predict the additional flow resistance to drain pipes due to the technique of laying drain pipes.

For the trenchless drainage technique the flow of soil around the body that is forced through the soil, is assumed to be a sequence of steady state situations in which the streamlines are the sliding lines to be described according to Prandtl. From the such known stream lines and an assumed displacement, a relation is derived between the deformation of the soil, the friction angle and the distance to the body.

A relation between hydraulic conductivity and deformation was derived by means of cores placed in a triaxial apparatus. When the friction angle is known, the reduction in hydraulic conductivity and the change in radial flow resistance can be determined.

For the traditional drainage technique, the change in dry bulk density due to settling of the loose soil with which the trench is filled up is estimated from the relation between change in density and density of the original soil in situ. From a relation between density and hydraulic conductivity the reduction in conductivity due to the change in dry bulk density can be obtained. The change in radial flow resistance then can easily be calculated.

The predicted hydraulic conductivity in the deformed zone respectively in the trench in a silt loam and a silty clay loam (marine deposits) was compared with the hydraulic conductivity in these zones derived from measured drain discharges, heights of water table and known hydraulic conductivities in the various, undisturbed layers. There appeared to be a reasonable agreement between the predicted and measured hydraulic conductivity.

Paper 3.12

APPLICATION OF OPERATIONAL RESEARCH DURING THE EXECUTION OF DRAINAGE SYSTEMS

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This article will be published in the Proceedings of ICID 10th Congress, Athens 1978, and therefore only an abstract follows.

Abstract

The operational aspects of subsurface drainage in the IJsselmeerpolders in The Netherlands are analyzed with the help of a network. Further attention is paid to investigations of installing a subsurface drainage system. Especially the activity "laying pipes" has been followed in detail to collect data for optimizing execution and planning purposes in general. For planning costs of the whole subsurface drainage system these data are very valid. A specification of drainage costs is given at the end of the report.

Introduction of this type of operational research may be a good help in tropical areas for large-scale irrigation projects combined with composed drainage system.

Group 400

Drainage of irrigated lands

CONTENTS

Paper 4.01

DRAINAGE INSTALLATION PROBLEMS IN THE SAN JOAQUIN VALLEY, CALIFORNIA, USA

William R.Johnston,
Westlands Water District, Fresno, California, USA

Summary

The area of the San Joaquin Valley, California, U.S.A., which has a subsurface agricultural drainage problem is described, as are the proposed solutions to the problem. The rate in which drainage problems have developed over the past ten years is tabulated for the 600,000 acre (240,000 hectare) Westlands Water District.

The costs and benefits of subsurface agricultural drainage in the San Joaquin Valley are described and compared with expected crop yield decrements due to not providing farm drainage after the ground water rises within five feet of the ground surface. Estimates indicate that the average benefit from drainage is about $ 150 per acre ($ 375 per hectare) which is approximately five times greater than the annual cost of a drainage system.

Specific problems pertaining to drain envelopes and pipe materials; trenching; and backfilling during the installation of subsurface drainage systems in unstable soils with an extremely high ground water table are also discussed.

The area

The great central basin of the State of California is one of the most intensively farmed and productive agricultural areas of the world. The southern half of this basin, the San Joaquin Valley, as shown on Figure 1, comprises about eight million acres (3.2 million hectares) most of which are devoted to irrigated agriculture. About 20 per cent of the irrigated area which is located in or near the Valley trough now has, or is projected to develop, high water tables and saline ground water (6,000 to 15,000 mg/l total dissolved solids) conditions sufficient to damage or destroy agricultural productivity.

On-farm drainage systems which are necessary to alleviate this problem were initially installed in parts of the Valley during the early 1940's,but system installations have been limited to about 50,000 acres (20,000 ha)

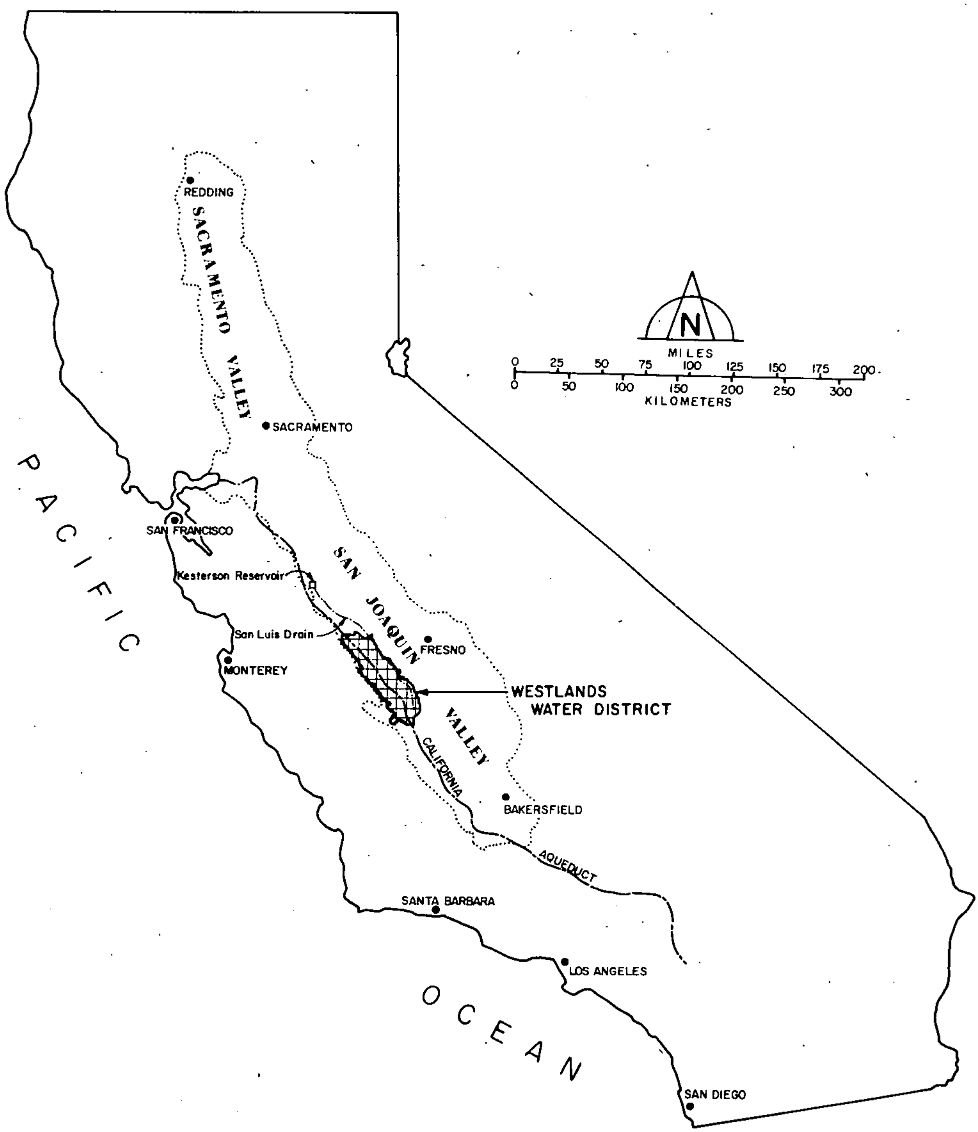


Figure I. California, U.S.A. Showing location of San Joaquin Valley and Westlands Water District.

because of the lack of adequate drainage disposal facilities. Discharge and recirculation of saline drainage effluent to irrigation canals or natural channels degrades the water for downstream reuse.

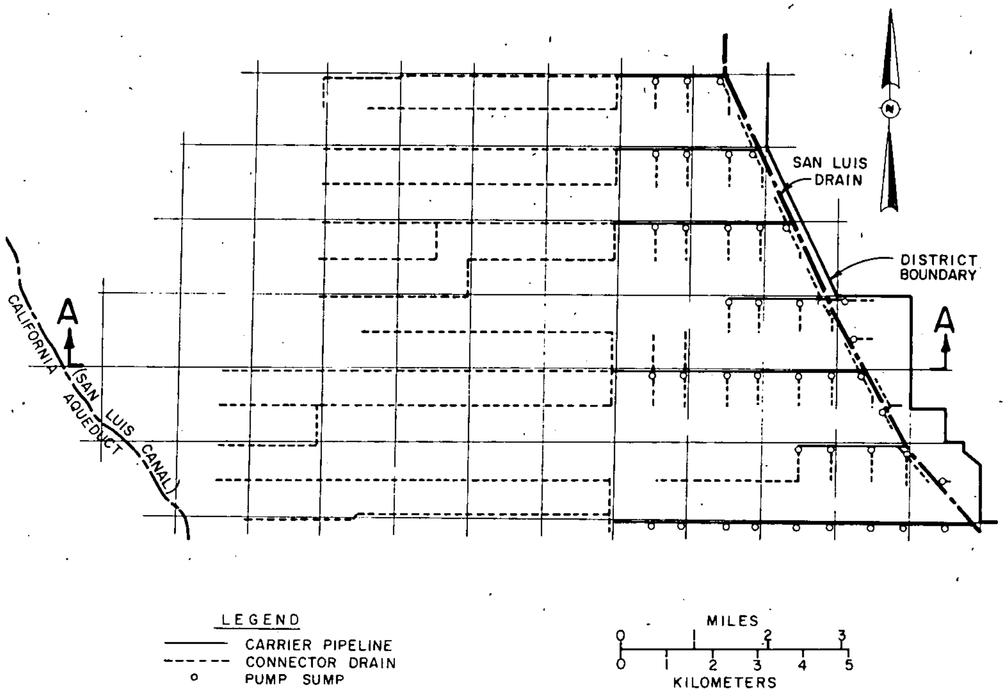
The projections of expected future conditions in Table 1 show that, within the next 50 years, almost 900,000 acres (364,000 hectares) of irrigated land in the San Joaquin Valley will require subsurface drainage in order to maintain a high level of agricultural productivity. The soils in the Valley which require drainage are generally stratified heavy clay and silty clay alluvium, basin rim and basin soils - all of which have a subsoil that is rather unstable when saturated.

TABLE 1. San Joaquin Valley drainage projections in thousands*

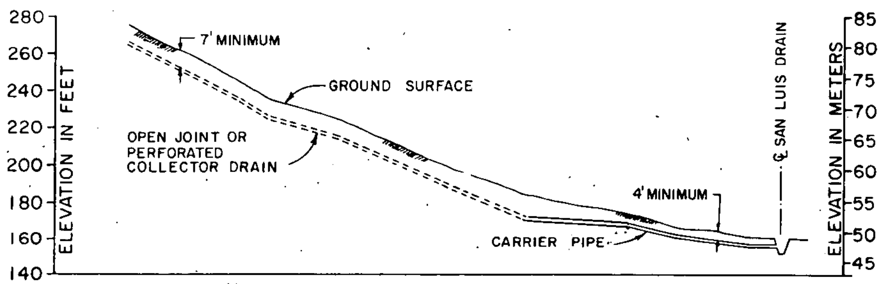
YEAR	AREA REQUIRING DRAINAGE		AREA DRAINED		ANNUAL AMOUNT OF DRAINAGE EFFLUENT	
	acres	ha	acres	ha	acre-feet	m^3
1980	400	162	56	24	60	74,000
1990	562	227	211	85	189	233,000
2000	687	278	336	136	276	340,000
2010	786	318	438	177	332	410,000
2020	868	351	521	211	366	451,000
2030	931	377	593	240	407	502,000
2040	982	397	651	263	425	524,000
2050	1,021	413	703	284	459	566,000
2060	1,053	426	746	302	486	599,000
2070	1,078	436	788	319	513	633,000
2080	1,097	444	823	333	535	660,000
Ultimate	1,142	462	941	381	606	747,000

* *San Joaquin Interagency Drainage Program, Progress Report No. 2, August, 1977*

The "Area Requiring Drainage" is defined as that area where a perched watertable occurs within five feet (1.5 meters) of the ground surface. Most of the 1,142,000 acres (462,000 hectares) that are expected to ultimately require drainage presently (1977) have a watertable within 20 feet (6.1 m) of the ground surface. It is assumed that some of the area requiring drainage will never be drained directly with an on-farm drainage system because of individual landowners on-farm practices, preferences, and economics. However, it is estimated that about 75 per cent of the area will eventually be drained in order to maintain a stable agricultural economic base. The increasing need for food and fiber will place a greater importance on maintaining the long-term productivity of such lands. Inasmuch as the drainage



TYPICAL PLAN



SECTION A–A

Figure 2. General layout of the Westlands Water District Drainage Collector System.

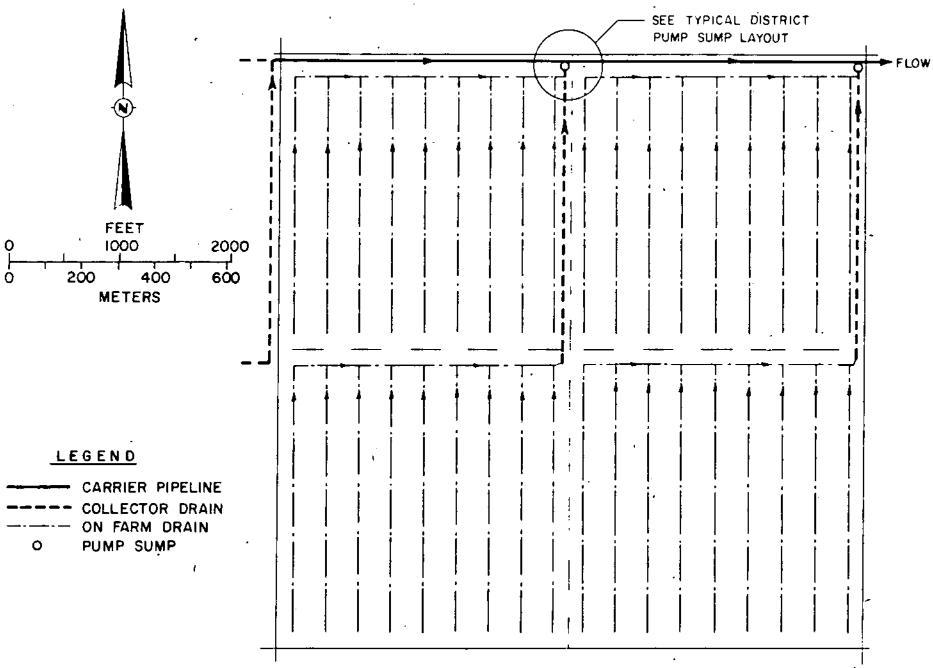
problems within California span a large area which traverse several levels of government, irrigation districts, drainage districts, and other political subdivisions, it becomes obvious that the need to ultimately collect and dispose of 500,000 to 600,000 acre-feet (617 million to 740 million cubic meters) of saline drainage effluent annually will require a major coordination effort between all of the drainage problem areas.

Westlands Water District

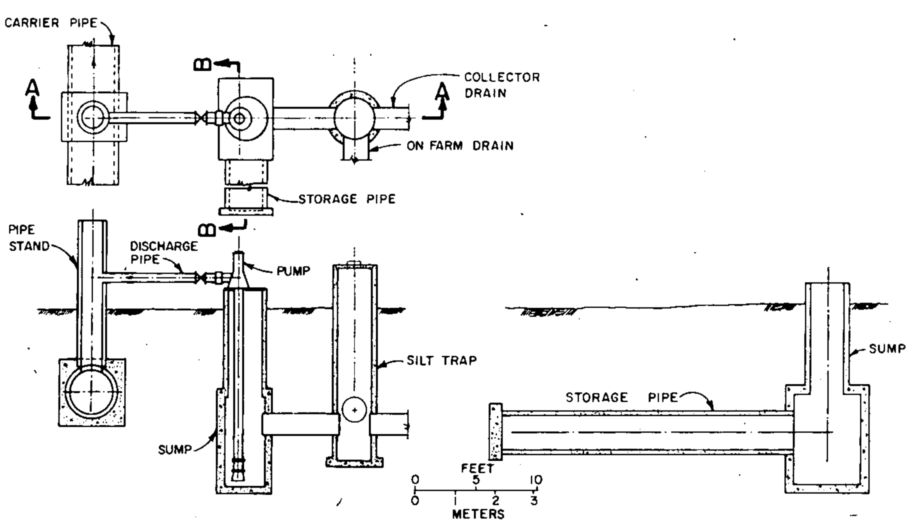
Westlands Water District, a political subdivision of California, covers approximately 600,000 acres (240,000 hectares) of prime farm land in the San Joaquin Valley between the foothills of the Coast Range on the west and the Valley trough on the east, as shown on Figure 1, is a part of this larger Valley drainage problem area. It is estimated that about one-half of the Westlands Water District area will ultimately need subsurface agricultural drainage facilities. Currently, more than 200,000 acres (80,000 hectares) of land within the District have a perched watertable that is less than 20 feet (6.1 meters) below the ground surface. The following tabulation shows the growth of the areas affected by the perched watertable in the District.

Depth to Water		Oct. 1967		Oct. 1976		Difference	
feet	*m*	*acres*	*ha*	*acres*	*ha*	*acres*	*ha*
0-5	0-1.5	12,300	4,980	34,900	14,120	+22,600	+9,140
5-10	1.5-3.0	57,000	23,070	81,200	32,860	+24,200	+9,790
10-20	3.0-6.1	89,400	36,180	89,100	36,060	- 300	- 120
		158,700	64,230	205,200	83,040	+46,500	+18,810

The above data shows that between 1967 and 1976 there was a 46,800 acre (18,930 hectare) increase in the area with a watertable of 10 feet (3 m) or less. The Bureau of Reclamation, a Federal government agency, is presently constructing an open outlet channel called the San Luis Drain to transport saline drainage water from Westlands Water District for initial disposal to the Kersterson (holding and regulating) Reservoir. Eventually, the drainage water will either be diverted from the Kesterson Reservoir to the western Sacramento-San Joaquin Delta-San Francisco Bay estuary by gravity or will be disposed of by pumping over the Coast Mountain Range to the Pacific Ocean.



TYPICAL ON FARM DRAINAGE SYSTEM
640 ACRES (160 HECTARES)



SECTION A-A

SECTION B-B

TYPICAL DISTRICT PUMP SUMP LAYOUT

Figure 3. Typical on farm drainage system and pump sump layout.

If found feasible, the saline drainage water will be used in wildlife management and/or for cooling electric generating plants before final disposal.

Westlands Water District has assumed the responsibility to collect the saline subsurface drainage effluent from each farm and transport it to the Bureau of Reclamation's San Luis Drain. The District's Drainage Collector System, which is currently under construction, consists of open-joint collector drains and closed carrier pipelines. The District's drainage system functions both as subsurface drain lines as well as a carrier of subsurface drainage water from the privately owned on-farm drain systems by gravity to the concrete-lined San Luis Drain. The collector drains discharge by gravity into closed pumped sumps or directly into the carrier lines.

One one-farm drain connection to the District's Drainage Collector System is permitted for each 160-acre (65 hectare) parcel. The connecting point is sufficiently deep to allow the on-farm drains to be 6 feet (2 m) deep and still discharge into the District drains by gravity.

Figure 2 and Figure 3 illustrate the general layout of the Westlands' Drainage Collector System and a typical layout of an on-farm system.

A contract to install about 115 miles (185 kilometers) of the District collector drains and 30 miles (48 kilometers) of carrier pipelines was awarded in June 1975. However, after installing the carrier pipelines and only 36 miles (58 kilometers) of collector drains, the contractor abandoned his work because of difficult field conditions and installation problems typical for the Valley soils.

These problems will be discussed in this paper.

Economics

Drainage costs

The cost of a subsurface drainage system will, of course, vary with the particular needs of an area. The average cost of plastic tubing installed for on-farm drainage with a gravel envelope, at depths of approximately 7 feet (2.1 meters) in the San Joaquin Valley is as follows:

Drain Diameter		Unit Cost in Dollars (U.S.)	
Inches	*Centimeters*	*Foot*	*Meter*
4	10	0.90	2.75
6	15	1.50	4.60
8	20	2.00	6.10
10	25	3.25	9.90
12	30	4.00	12.20

In order to compare the costs and benefits for agricultural land in the San Joaquin Valley, a typical drainage system in Westlands Water District will be used as an example.

A typical on-farm drain system, shown in Figure 3, contains approximately 150 feet of drain per acre (45 meters of drain per hectare). The costs of such a system would be approximately $160 per acre ($400 per acre). If it is assumed that such an on-farm drainage system can be financed for a period of 20 years at an average interest rate of 9 per cent, the annual equivalent capital cost is approximately $17,50 per acre ($43.75 per hectare). The cost of the District's collection facility is estimated to be approximately $300 per acre ($750 per hectare) served when completed. The District has obtained a 40-year interest-free loan for the construction of the drainage collection system. The equivalent annual cost for the District's drainage system is approximately $7.50 per acre ($18.75 per hectare). The District also must pay the United States Government $0.50 per acre-foot ($0.41 per 1000 cubic meters) of irrigation water delivered for the use of the San Luis Drain and other related facilities. With an average water requirement of about 3 acre-feet per acre (9250 cubic meters per hectare), the annual drainage disposal cost will be about $1.50 per acre ($3.75 per hectare). Assuming the operation and maintenance of the District's Drainage Collector System and the on-farm drain systems costs are $3.50 per acre ($8.75) per hectare) the estimated total annual cost will be about $30 per acre ($75 per hectare).

Benefits of drainage

When a saline ground watertable rises into the crop root zone, a decline in crop yield can be expected. This decline is due both to the high watertable and the resultant increased salinity in the crop root zone. The range of expected decline in crop yield, when no on-farm drainage is provided, is shown in Figure 4, which compares the per cent of relative crop yield decrement to time after saline watertable raises to within 5 feet (1.5 meters) of the ground surface.

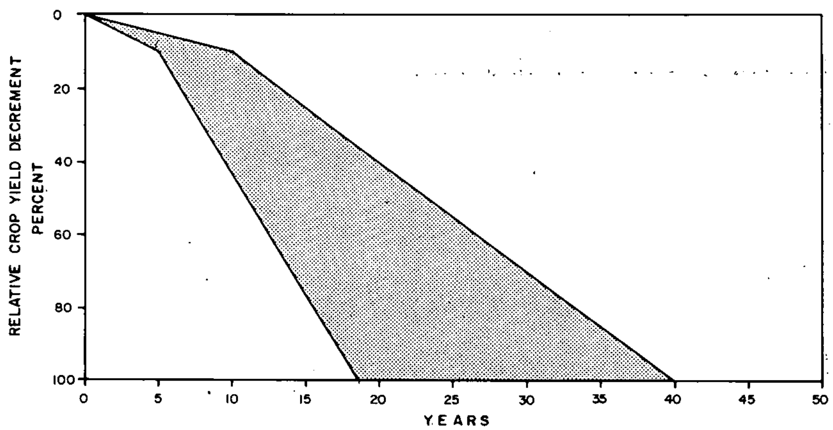


Figure 4. Expected crop yield decrement due to no on farm drainage provisions after ground water raises to a minimum of 5 feet (1.5 meters) below ground surface.

It is quite apparent that if drainage is not provided, when needed, the crop yield will be reduced by as much as 20 per cent in a very few years. It would quickly be uneconomical to cultivate land with this reduced productivity. It could be assumed that the agricultural value of the land would soon be greatly reduced or lost entirely.

Obviously, the landowner or farmer receives the primary benefit of the continued productivity of the soil and the sustained land values resulting from the timely installation of a needed drainage system. This direct annual benefit from the investment in irrigated agriculture should, in fact, be equal to the rate of return for an equal capital investment elsewhere and should probably approximate 10 per cent of the value of the land and improvements when it is producing a maximum crop yield. Since the present value of the agricultural land in the Westlands Water district averages approximately $1500 per acre ($3750 per hectare), the annual drainage benefit can then be estimated to be $150 per acre ($375 per hectare). This indicates that the primary benefits received by the landowner from the timely installation of a complete drainage system that has an annual equivalent cost of $30 per acre ($75 per hectare) are five times greater than the annual cost of the system. This amounts to an annual rate of return on the drainage investment of approximately 30 per cent.

Additional benefits are realized by local and regional economies by having sustained employment levels and maintenance of a stable economic base. Also, because of this continued farm production, the farm suppliers and manufacturers of farm-related equipment and machinery will enjoy full employment and constant industrial production while satisfying the material and equipment needs of the farmer. The farm product processor is also able to maintain a high level of employment and plant capacity in handling and preparing the food and fiber for shipment to the merchant and the consumer. The consumer is benefited by a continued supply of food and fiber at reasonable prices when commodities are grown locally or domestically in abundant quantities. When the consumer's demand for any commodity exceeds the available supply, goods are transported long distances or imported - usually at higher prices to satisfy the demand. Increased imports to satisfy the commodity needs of a nation can contribute to foreign trade deficits. These deficits, if prolonged, will eventually affect the stability of a nation's currency. Therefore, the beneficiaries of sustained agricultural productivity through the installation of on-farm drainage systems are farmers, farm suppliers, processors, manufacturers, merchants and, last - but not least - consumers.

Material problems

The need to provide subsurface drainage for an extensive area of farmland in the San Joaquin Valley of California and many other areas of the world to protect agricultural productivity means that it is most important to design and construct the best drainage facilities possible. Good construction is necessary to maximize the life of the facilities and to minimize maintenance problems once the facilities are completed. Even though there have been many recent improvements in regard to the type of materials and equipment used in the installation of subsurface agricultural drainage facilities, there are still many problems needing research and development work to assure the installation of an economical, efficient, trouble-free drainage facility - particularly under adverse construction conditions.

Pipe

There are three basic types of pipe used for subsurface drain construction in the San Joaquin Valley: 1) concrete pipe, 2) clay tile, and 3) corrugated plastic drain tubing. For many years concrete and clay were the predominant materials used to construct drain conduits.

One problem with concrete is that under high sulfate soil and water conditions corrosion of porous concrete drain tile can take place. The most prominent among aggressive salts which affect concrete in the San Joaquin Valley and many other arid areas of the world are sulfates of sodium, magnesium and calcium. These salts, which are sometimes known as white alkali, react chemically with the hydrated lime and hydrated calcium aluminate in cement. These reactions are accompanied by considerable expansion and disruption of the concrete. Dense concrete containing cement which has a low content of tricalcium aluminate is highly resistant to this sulfate attack and has been used widely in the manufacture of concrete pipe installed in drainage facilities. Good quality clay drain tile are, for all practical purposes, immune to the corrosive action of soil and water sulfates. However, clay drain tile have recently become relatively scarce and expensive.

Today, the most small diameter drain conduits are flexible plastic corrugated tubing. Plastic tubing appears to be well suited for agricultural drainage when properly manufactured and installed. However, there are certain problems in its manufacture that still need attention. The latest developments in tubing design provide suitable openings for water inflow and adequate structural strength to withstand the forces placed on the tubing when properly installed to depths of 8.0 feet (2.4 meters). It is still not uncommon, however, to have tubing delivered to the field with excessive variation in wall thickness. This minimizes the structural strength of the tubing and leads to collapsed drains. In addition, tubing seam ruptures, cracks or splits result in poor drainage installations.

Envelope

Two types of envelope materials are generally used in agricultural drainage at various locations around the world -- natural graded gravel or sand materials and synthetic fabrics of several types. In many areas the use of an envelope material does not appear to be necessary for good drainage. Where a gravel envelope is required, more often than not, it is needed to facilitate the drain installation and to provide a structural base for the drains. It is also beneficial in reducing the convergence of flow as the ground water moves from the less permeable saturated soil into the more permeable gravel envelope material and then into the perforations or openings in the conduit.

Gravel and sand drain envelopes are used extensively in the San Joaquin Valley for the above reasons and because of the heavy unstable soils in which the drainage problems occur. However, the high cost and, at times, the unavailability of the native sand or gravel make it imperative that an adequate substitute be developed. To date, none of the synthetic envelope materials available provides the stability needed in adverse installation conditions, but some show promise as "filters".

Installation problems

Both trenching and trenchless installation techniques are used to install on-farm drainage facilities in the San Joaquin Valley of California. Even though there are more trenching machines used because of their versatility, there are some serious problems related to the difficulties of installing good drainage systems with either technique.

Trenching and backfilling

Chain type trenchers, in optimum soil conditions, have a greater digging capacity than bucket type trenchers. The bucket type trencher is more versatile in some of the adverse soil and water conditions that are encountered during the installation of subsurface drains. Some clay soils become quite sticky when the water content is near saturation. The sticky clay adheres to the chain of the chain type trenchers, stopping the trenching operation. It then becomes necessary to decrease the stickyness of the clay soil being excavated by spraying water on the chain and raising the water content of the soil in contact with the chain. The spoil (excavated material), under these conditions, is little more than a slurry and becomes almost unmanageable, particularly during the backfill operation. If it is placed in the trench immediately, it will dry as a mass. If it can be allowed to dry before backfilling the trench, then it is like trying to fill the trench with large bricks. It is questionable if either situation promises the installation of a good drainage system. Bucket type trenchers usually perform better under adverse digging conditions.

In unstable soils, it is usually important to backfill the trench immediately behind the trenching machine during the placement of the drains to insure that the drains remain on line and grade. This is especially important when the trench extends into saturated soil and the drain is being placed below the watertable.

The weight of the soil adjacent to the excavation bears on the soil stratum at the level of the bottom of the excavation; and, if the bearing capacity of the saturated subsoil is not great enough to support the weight, the bottom of the trench will bulge upward, causing the drain to be pushed off grade. When the trench extends into sand, the bottom is ordinarily stable as long as the water level inside the trench is no lower than the ground water outside the trench. However, as soon as the water inside the trench is lowered by the installed drain, an upward seepage of water is created. If the difference in water levels is excessive, the bottom will heave, then become quick, and sand boils will appear. Gravel envelope material can be lost in these sand boils, as its weight is insufficient to stabilize such material. To adequately stabilize this condition, it is sometimes necessary to use gravel as large as 3-inch (7.5 cm) in diameter as a drain envelope to stabilize the trench. The problems just described were all encountered by the contractor when installing 10 to 24-inch (25 to 61 cm) diameter concrete pipe at 8.0 to 12.0 feet (2.5 to 3.7 meters) depths for the Westlands' Drainage Collector System with a very large and heavy wheel type trenching machine.

Trenchless installations

The trenchless method of installing drainage tubing requires a much higher draft force than the trencher method of installing drains. These forces rise sharply as the width of the shank and the depth of the installation increases. Therefore, drains installed by this method are limited to 4-inch (10 cm) diameter and a depth of approximately 8.0 feet (2.5 meters).

Envelopes

When installing a gravel envelope around corrugated plastic drain tubing, the placement of the gravel in the trench in relation to the bends in the tubing appears to be an important factor. Temporary stretching of the tubing occurs where the tubing is bent to make the vertical turn at the bottom of the trench. The corrugations on the top of the tubing are compressed and those on the bottom are expanded during such curvature. If the expanded corrugations are filled with the envelope material as the machine makes forward progress and the tubing becomes parallel to the bottom of the trench, the top portion of the tubing must also expand since the lower portion cannot return to its original shape if the corrugations are full of gravel. The timing of placement of the gravel envelope is, therefore, quite critical. Gravel must not be placed around the tubing until it is on proper grade and alignment. It appears that when a synthetic fabric envelope is placed around the tubing before the gravel envelope is placed around the tubing, the synthetic envelope will eliminate this problem because the gravel envelope material is prevented from moving in between the tubing corrugations.

Another problem occurs when insufficient amounts of gravel envelope material or a synthetic envelope is used without gravel. The tubing, being buoyant, has a tendency to float when submerged prior to the trench being properly backfilled. This can cause problems in both alignment and grade, and may greatly reduce the efficiency of the drain line. The synthetic envelope materials presently available, while capable of acting as a filter, do not provide adequate weight to overcome the buoyancy of plastic tubing prior to backfilling. Under these conditions the trench must be backfilled simultaneously with the placement of the drain and envelope material.

When a subsurface drain is installed with trenchless equipment, the placement of the gravel envelope can be hampered when the soil is saturated and the resultant hydrostatic head is high. The opening in the soil created for the tubing and the gravel envelope will rapidly close as the semi-fluid soil tries to flow into the opening, thus restricting the flow and proper placement of the gravel. The severity of this problem has been decreased by one contractor in the San Joaquin Valley by placing a power auger in the opening around the tubing, which forces the gravel into the open space provided for it, thereby overcoming the hydrostatic pressure of the soil.

Tubing stretch

Polyethylene tubing will stretch under tension during installation. Excessive stretching can cause transverse slots in the tubing to become quite wide, thereby allowing the gravel envelope and native soil to migrate into the tubing, causing the drain to become blocked. Stretching of the tubing also decreases its resistance to deflection. Stretching appears to be attributed to several other factors in addition to improper placement of the gravel envelope material. One such item is having the drainage machine speed exceed the rate at which the gravel envelope material will flow freely into the trench around the drain line. Another is using crushed rock that will create more friction than well rounded gravel and cause tubing to stretch more at the same installation speed.

It is essential that the amount of stretch be minimized and one way to know the amount of stretch that is occurring is to lay the tubing on the ground ahead of the drain trencher, rather than to carry it on a reel on the machine. However, in hot weather the temperature of black polyethylene tubing increases as it lays in the sun. The tubing then has much less resistance to stretching and its resistance to deflection can be reduced up to 50 per cent when the temperature of the plastic reaches the vicinity of 120° to $140^{\circ}F$ (50° to $60^{\circ}C$). The tubing stretch can be substantially reduced or eliminated by the installation of a pair of tubing drive sprockets along with the proper use of gravel envelope installation methods. If the perimeter of the prockets is driven at the same rate of speed as the forward progress of the trencher, stretch of the tubing is eliminated.

After the proper materials and installation techniques for the anticipated construction conditions are selected, a rigid inspection program must be carried out during construction. Many maintenance problems can be attributed to the careless attitude of a workman and it is essential to strive for a quality drainage system, rather than try to compromise with some lesser degree of workmanship.

Maintenance

Crop roots can be a problem even in drains placed to 8-foot (2.5 m) depths. Plant roots that enter drains in sufficient quantities cause plugging. Root plugging problems are minimized in the San Joaquin Valley by periodically blocking the outlets and injecting a copper sulfate solution through an air vent in the end of each drain line and allowing the solution in the drain for two to four days before placing the line back into service.

In areas where magnesium or iron oxides are present, incrustations of these materials can occur and close the perforations or openings in the drain lines. These incrustations can be removed by injecting a 2 per cent solution by weight, of sulfur dioxide and water through a vent or riser at the upstream end of the drain.

The drains must also be sealed for a period of two to four days so that the contact time is adequate to allow the sulphur dioxide solution to dissolve the deposits. Care must be taken, however, because the relatively unstable sulfurous acid (H_2SO_3) also reacts with any available oxygen as well as with metals to produce varying amounts of sulfuric acid (H_2SO_4), which will dissolve some synthetic envelope materials. A comparison of drainage flows prior to and after treatment will provide an estimate of the benefits from the cleaning operation.

The effluent discharged from the drains following any chemical treatment will contain chemical residues, acid in varying states of dilution and water with a very low dissolved oxygen content. The effluent should be monitored and diluted if necessary to prevent damage to fish and wildlife and/or to prevent problems for any potential downstream use of the water.

Conclusion

The materials and installation methods used in the construction of subsurface drainage systems on agricultural land have improved substantially during the past decade. However, research and experimentation must be continued to develop alternative materials and to improve equipment, so that agricultural drainage systems can continue to be provided within the economic means of the farmer. It has been demonstrated that the proper and timely installation of drainage systems will sustain agricultural productivity, which benefits everyone.

There does appear to be several general areas where continued improvement is needed, particularly in the design and production of materials for, and the construction of drainage systems. They are:

1. The specifications and standards necessary to produce good materials and installation procedures;

2. The quality and uniformity of the manufactured materials; and

3. The reliability of contractors or workmen building drainage systems.

In addition, two specific problems relating to the installation of drainage systems in the San Joaquin Valley of California need additional attention. They are:

a) The economic production of a synthetic envelope material which will serve as a suitable replacement for graded gravel or sand; and

b) The manufacture of a drainage machine suitable for installing large size 12 to 21-inch (30 to 53 cm) drains in high watertable conditions.

Paper 4.02

DRAINAGE KNOWLEDGE AND RESEARCH NEEDS IN DEVELOPING COUNTRIES

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Summary

As the pressure to grow more food increases worldwide, so does the need for drainage and, in turn, the demand for information and technology. The present status of knowledge and technology is considered generally adequate to meet short-term needs of drainage design and implementation in developing countries. The disappointing results of many newly irrigated and drained areas should not primarily be attributed to an insufficient data base but, rather, to a lack of understanding of the broad interrelationships between the various technical components and of the role of institutional factors. Therefore, the development of new and advanced theories and methodologies is not of immediate concern. It is the application of what is available that is inadequate. Thus, short-term research priorities should be directed toward constraints in the transfer of information. Since most technical information is generalizable to a fair degree, these constraints are primarily in the adaptation and acceptance of technology and concepts. Adaptation related to irrigation practices and related activities which govern drainage systems in dry areas rather than to drainage technology itself. Acceptance, or adoption, refers to the farming community as well as to higher decision-making levels.

Long-term research needs cover a much wider area. In fact, they relate to most aspects of drainage investigations, rationalization of designs and implementation. In addition, long-term research should be increasingly directed to broad optimization programmes of water management.

Effective research programmes at the local level call for national as well as regional and interregional coordination, guidance and support.

1. Increasing scale of drainage

Drainage programmes are on the increase in both the humid and arid regions of the world, whether in high-income areas or developing countries. The objectives are the same in all cases - to protect the quality of the land and to grow more food. In traditional drainage countries, such as The

Netherlands, United Kingdom, Germany, and the United States, old drainage systems are being replaced, or remodelled, with systems that are more finely attuned to the requirements of present-day intensive crop production. According to the American Soil Conservation Service, about 30 per cent of the USA's "Prime farmlands", or 110 million acres, is land with a natural wetness problem that has been solved by drainage. FAO (1977) estimates that 52 million hectares of irrigated land in developing countries will need to be drained in the 1975-1990 period to control waterlogging and salinity. In the same period some 26 million hectares of rainfed land will profit from the improvement or introduction of simple on-farm drainage systems. The total cost is estimated at about 14,000 million dollars.

Countries that have only recently embarked on substantial drainage programmes are found primarily in regions where water development schemes have resulted in the increased availability of irrigation water. This, in turn, has led to higher deep percolation losses from canals and on-farm application resulting in rising groundwater levels. It is clear that most of the world's irrigated land will eventually be in need of watertable control to prevent waterlogging and soil salinzation. At present large-scale activities are being developed in such Near Eastern areas as Iraq's Mesopotamian Plain, Egypt's Nile Delta and Valley, Syria's Euphrates Valley, Pakistan's Indus Basin and Iran's Khusistan Basin. Drainage problems are also developing in irrigated areas in Hungary, Roumania, California's San Joaquin Valley, Arizona's Salt River Valley and Mexico's Mexical Valley.

New drainage activities are not, however, limited to irrigated land; they are also found in the wetlands. In North Africa, Latin America, India, Korea and many others, surface and subsurface systems are planned and implemented as pressure for higher production on rainfed land grows.

Though recent drainage programmes in developing countries are of considerable magnitude, there are still vast areas that are in the process of degradation due to waterlogging, salinity and erosion. In practice, drainage is often considered a necessary evil in irrigation projects and, worse, it is left out of many irrigation schemes or postponed till an unspecified later date, which is usually after serious damage has already been inflicted. Drainage is often too expensive for traditional production patterns in developing countries and requires changes that deeply affect the farming community. In the institutional field, numerous arrangements are needed to adjust irregular patterns and shapes of holdings, bring in needed expertise, organize and initiate research and maintenance programmes, etc. In spite of these and similar constraints, indications are that drainage is being increasingly looked upon as an integrated part of the modern package of tools in agricultural production, and as a must if land is to be cropped to its productive capacity.

2. Needs of technology and expertise in developing countries

As drainage programmes grow in number and scale, so does the need to make site-specific drainage criteria and design parameters available in a timely manner. Since most developing countries are not traditional drainage countries, local experience is limited and there is not sufficient time and manpower to build up this expertise through research, experimentation and gradual application on practical scales. It is common practice, therefore, to use concepts, criteria and technology from areas that are similar to the project conditions. Testing their local applicability is usually initiated but insufficient research coordination, shortage of competent staff and changing emphasis in research elements often lead to premature termination of research programmes and inconclusive results.

When transfer of technology has been carried out, the benefits of drainage in terms of improved soil moisture conditions, reduced soil salinity and better crop yields are often disappointing. Thus the question arises as to what extent this may be due to the inadequacy of the available technology or to problems in transfer. It is noted, in this connection, that drainage and water management projects in developing countries are mostly located in areas that are characterized by traditional cropping patterns on a subsistence basis, by a need for improvement in farmers' skills and understanding of water management; and furthermore, by weakly developed institutions such as extension services, limited credit facilities, fertilizer availability, seed quality and storage capacity. To obtain higher production, improvements must be made in all these fields, not only in drainage and water management.

Therefore, we should expect crop yields and capital intensity of farms to increase only gradually over the years. The sensitivity of production to refined control of surplus water will increase correspondingly. For the time being, however, this sensitivity is relatively low and some flexibility in the precision with which drainage design parameters and criteria are established is considered acceptable. It should be expected that adjustments in the drainage intensity will be introduced in the course of the years, as the level of water management increases and agricultural output expands.

3. Status of knowledge and technology

An analysis of the present state of affairs regarding available information and knowledge will imply the selection of a criterion to assess their adequacy. Within the context of the FAO's development programmes and objectives such a criterion is unlikely to be of a strict academic nature. Rather, the question is whether information that has been developed over the years in the world as a whole satisfies present short-term needs of improvement and development projects. This question of short-term adequacy is briefly examined below for a number of drainage problems and elements. No attempt has been made to be complete or to deal with the details.

It is shown that the technology and the information are available in sufficient quantity to satisfy short-term needs of developing countries, i.e. to make rational designs for most wet and salty areas. However, a substantial number of newly irrigated and drained areas has been less successful than hoped for, and there have been outright failures. It has often been claimed that these projects have been designed on the basis of inadequate information and that the result would have been better had more time been allocated for further studies. There are strong indications, however, that this is not necessarily so in all cases and that, in fact, insufficient or insufficiently precise data is seldom the real cause of the problems. Rather, it is unawareness of the effects that changes in the water regime, brought about by water development and irrigation, may have. It is also a lack of understanding of the broad interrelationships between the various technical components, as well as between these and non-technical factors. Amongst these latter, the impact of deficient farmers' skills and institutional arrangements for on-farm water use is of great importance. Most of the problems presented so far can be avoided if the right questions are asked at the right time.

3.1 Drainage theories

The theories of groundwater and surface waterflow are well established for a wide range of conditions and are universally applicable. In fact, equations are available which are considerably more precise than the data on the soils and hydrologic conditions that are used in the calculations. Difficulties do occur in the application of theories, but these stem mostly from the inability to simplify heterogeneous field conditions to known models. Only in special situations will there be a need to conduct research at the base prior to project design.

3.2 Investigation methods

Field and laboratory methods to survey and analyse local project conditions, in particular soils, hydrogeology and climate, are known and available. They are insufficient to diagnose the problems and to design solutions. The information developed, however, is frequently not as precise as would be desirable from a design point of view. This is due to the heterogeneity of the conditions, particularly of the soils, rather than to the investigation methods applied. A greater density of the observation network will usually lead to higher precision. This, however, is costly and the network density is limited for practical and economic reasons. An initially lower level of precision in pre-design investigations may be acceptable if, through the introduction of drain performance tests on sample areas of several acres, additional information can be developed, and so improved designs obtained during the first part of the construction phase. In so doing the project implementation may be considerably advanced.

3.3 Criteria and design parameters

Agronomic criteria for drainage refer to the water regime that should be effected in the rootzone of the crop. In practice, they relate to depth, duration and frequency of water ponded on the ground surface and to the position and fluctuation of the watertable. For groundwater, more specifically, they refer to watertable levels that are not to be exceeded, or that may be exceeded over specified distances and duration and with specified frequencies. Clearly, the criteria are highly dependent on crop, growth stage, season and soils. They are also closely linked to the amount and frequency of damage that is economically acceptable.

The available information is considerable for both humid and irrigated areas and this permits us to express the criteria in a quantitative form for most conditions. However, crop response cannot always be predicted with a high measure of precision and opinions differ on specific values. As an example, some engineers consider it acceptable that the watertable rises into the upper 50 to 100 cm after each irrigation, for periods of 2 to 5 days. Others base designs on the criterion that the watertable should not exceed, in any part of the growing season, a level of 1.0 or 1.5 metres below the ground surface. In humid areas with storm rainfall and resulting watertables occasionally close to the groundsurface, the criterion is expressed as a rate of fall of the watertable. Opinions differ, however, on the optimum drop rate for different crops in difficult growth stages.

Obviously, more data is needed to further specify the criteria. However, it is felt that the existing information and expertise is sufficiently usable in the meantime, to permit the design of systems in short-term project development. This conclusion does not apply to such specific drainage problems as those of some tropical peat soils, acid-sulphate soils, paddy fields and tropical mud flats, information on which is particularly deficient.

These uncertainties of agronomic criteria as well as of the limitations in investigation programmes have a direct bearing on the determination of such design factors as the specific discharge rate, drain depth, leaching rate, capacity of drains and drain spacing. In addition human and institutional conditions will have a profound impact on water management and thus drainage design. A typical example is the drain depth which, in irrigated areas, is often set at 2 m for field drains. It might be agreed that, from a strictly technical viewpoint, a depth of 1.5 m would be acceptable. Shallower drains, however, bring about an increased sensitivity to deficient water management and, consequently, to a higher risk of waterlogging and salt accumulation. Present tendencies are for deeper rather than shallower drains, particularly in capital intensive cropping areas.

Another example is the design discharge rate in irrigated areas. This parameter is primarily influenced by deep percolation irrigation losses and by leaching requirements. The deep percolation losses, in turn, depend very much on such factors as system management and operation, farmers' understanding of irrigation and salinity, land preparation, methods of irrigation, functioning of users' organizations, etc. Most of these are hard to express in quantitative terms and, moreover, may show changes in the course of time. As a result, the design discharge rates that are selected in practice, if translated in a steady-state value, vary from about 1 to 4 mm per day, the difference being more closely linked to the concept and experience of the designer than to physical conditions.

It would not be difficult to make a long list of items on which available knowledge is inadequate. Such a list, however, is likely to refer to specific problems rather than to the conditions that are commonly found in development or rehabilitation projects. For these latter, the available information is considered adequate in a short-term context.

3.4 Drainage materials

The performance of drain pipes, whether made of clay, concrete or plastics, can be reliably predicted for most soil types. On drain envelopes, however, there are still questions to be answered, and conditions that require a filter are also not yet well defined. Granular filters can be designed to match the soil on the basis of existing knowledge. However, their cost may be high if sieving is needed. Most other filter materials do not match the soil and their performance cannot usually be predicted with complete confidence.

Since the interpretation of results of laboratory research to field conditions is difficult, field research may be indispensable in some new areas for safe technical and economic designs.

3.5 Installation methods

Pipe drains are mainly installed by trenching machines in irrigated areas, and by both trenching and trenchless machines in humid areas. Whilst trenching techniques are basically applicable anywhere, some engineers harbour reservations about trenchless techniques on layered soils if the discharge of water depends heavily on the hydraulic conductivity of the trench backfill. Questions have also been raised about possible compaction around the drain pipes, particularly in soils of higher clay content. It appears that more research is needed to enable designers to predict the performance of drains laid with trenchless techniques more precisely. Methods using trenching machines, on the other hand, are considered widely applicable.

The installation of tubewells for drainage has become a familiar technique in important new drainage areas which include large tracts in India and Pakistan. Problems of corrosion of well materials, particularly in areas of salty groundwater, have been partially solved through the introduction of such materials as fibreglass and asbestos cement. Here again, more researcn is needed but the available technology is adequate for the installation of tubewells in the short run.

3.6 Specific problem areas

What was said in the above refers to a kind of "standard" conditions in humid arid zones, which are characterized by the absence of specific problems. Examples of specific problems are the drainage of paddy fields, mud flats, tropical peats, acid-sulphate soils, and some vertisols. Though some information is available, considerable research may be needed in each of these prior to the design of optimum solutions.

4. Constraints in the transfer of technology

The application of existing knowledge on drainage to new areas has been and still is slow. Degradation of land through waterlogging, erosion and salinity is widespread and continuing. The processes and factors that play a major role in the transfer of information and technology may be grouped as

a) generalization, b) adaptation, and c) adoption.

4.1 Generalization

Most technical information is generalizable, i.e. it can be directly used in or adjusted to new conditions. As observed in the previous section, the drainage theories are universally applicable. The investigation's methods permit to identify and characterize the physical conditions to most areas with sufficient accuracy to predict drain performance. Most criteria and design parameters are site-specific. However, since there is considerable scope for interpretation and extrapolation, they may be considered as partially generalizable as well. This does not apply in full to drain envelope materials where performance cannot always be predicted with complete confidence.

4.2 Adaptation of technology

Available technology in water management is not often used by farmers because they believe it does not apply to their specific physical and other production conditions, the cost is too high, the required energy source is not available, the foreign currency component is too high, or credit facilities are lacking. Thus there is a need to adapt known technology to the specific physical and socio-economic conditions of a project area. This applies particularly to on-farm irrigation which is governing some critical drainage design parameters and which has several components in common with drainage systems. However, whilst on-farm irrigation systems, once constructed, are operated by farmers, drainage systems rarely are. Once installed, the drainage system needs only periodic maintenance by either the farmers or contractors. An exception may be where drains serve as supply ditches in the irrigation season or where surface drains need to be annually reconstructed. Consequently, appropriate technology in drainage has a different function and should primarily serve to reduce installation cost while maintaining quality and performance. As an example, drainpipe laying machinery has become rather sophisticated, but is commonly used since the alternative of execution by hand labour appears impractical and offers few economic advantages. The digging and maintenance of ditches, on the other hand, is still often being done by hand labour. Furthermore, clay or concrete drain pipes are normally being manufactured locally since this requires only simple and inexpensive equipment. Drain envelope materials are mostly found or produced locally as well as are auxiliary structures such as inspection pits, silt traps, drain outflow structures, etc. Simple tools and techniques are also of considerable importance in such surface drainage activities as land forming, and construction of small farm drains and control structures.

4.3 Adoption

Adoption of new drainage concepts and techniques by the local farming community as well as by project and higher decision-making levels, meets with constraints that are similar to those met in broader water management schemes. These constraints are institutional, legal and socio-economic in nature. Almost universally, farmers and administrators are reluctant to invest in drainage improvement as long as there are doubts about returns or as long as waterlogging is not noticeably present. To make drainage pay, traditional cropping patterns may need to be changed. Credit may not be easily available. Drainage normally requires a group of farmers to cooperate in a rather specific sense and perhaps sacrifice land. Comparatively little research has been done so far on the identification of specific factors that interfere with the rate of acceptance of drainage concepts and considerable gaps exist in knowledge and understanding in this area.

In conclusion, transfer of research results in drainage to development areas does not generally meet with major constraints of generalization and adaptation. However, adoptation of drainage concepts and technologies is still slow and research is needed to obtain improvement.

5. Research needs and priorities

5.1 Short-term research

It appears that it is not the development of new and advanced technology that is of concern in the short run. Rather, it is the application of what is known. Since most of the information is generalizable, a survey of the local project conditions will basically be sufficient to prepare the designs of the drainage systems. In practice there is often a need for field experiments or for performance tests to check the designs as well as to build experience and confidence. It was shown, however, that there are still specific topics and problem areas on which available knowledge is limited and which should be given priority in short-term research programmes. In this respect particular mention was made of drain envelope materials in relation to soil type and a number of special problem soils.

Considerable short-term research is also needed on the adaptation of technology, the development of appropriate technology, and on constraints to its acceptance by the farming community. Research emphasis in these areas should be on the factors that govern the need of drainage and drainage intensity rather than on the actual techniques of drainage. These factors are found particularly in irrigation practices.

With regard to the adoption of concepts and technology, research is needed into farmers' responses to constraints which result from the prevailing water codes and regulations, social organization patterns and local decision-making processes, problems of leadership and status patterns, problems of coordination and integration of institutions from the project formulation level down to farmers' organizations, the flow of information between research stations, advisory services and farmers. Research is also needed on the possible changes in the social system, skills, working and living conditions that result from irrigation and drainage improvement projects. These factors have not yet been subjected to intensive and systematic research at the required scale. A major, coordinated effort, which takes into account the rather unique position of drainage and water management in agricultural development would appear to be justified. This position is primarily characterized by the large, non-recurrent, capital investment needed for water systems and by the fact that water management facilities are largely beyond the control of the individual farmer.

5.2 Long-term research

It has become apparent that there are many issues - relating to almost all aspects of drainage - that require further study in the long run.

Research should be conducted on the further improvement of present methods and techniques as well as on the development of new technologies, methods and methodologies for the optimization of technical designs and the effective operation and maintenance of drainage systems. Some examples are: crop response to specified watertable fluctuations and soil salinity regimes in various types of soil; economics of drainage; methods of characterization of soils and deeper strata in relation to drainage; low cost methods of drain installation; effects of trenchless drain installation on permeability of soil around the drains; design criteria for vertical drainage in humid areas; design discharge rates in relation to rainfall patterns, soils, irrigation and salinity; and, the mechanism of salt transport in soils. These and related subjects deal with a further rationalization of technical drainage designs. Drainage research, however, should also - and increasingly - be oriented to broader programmes of optimization of water and agricultural management. In addition, long-term research is also needed on the transfer of technology which includes complex problems of a socio-political and legal nature, educational programmes, institutional models, etc., as related to on-farm water management.

6. Implementation of research

Most of the short-term research is site-specific and calls for work at the field and project level. Various programmes are at present being carried out in the countries that have important drainage and irrigation programmes. Their number, however, is too small to adequately cover the needs, and the methodologies used differ from one country to another and sometimes even from one area to another within the same country. Results, therefore, are difficult to compare and are not systematically collected and disseminated to other users. Feedback from the field to research centres is highly inadequate or often lacking altogether. Longer-term research is both site-specific and includes work at the base that could be done anywhere. Here again, however, new development projects would benefit considerably from a systematic and coordinated effort.

It appears that action is needed at both local and international levels. At the local level there is a need to involve research institutes and university faculties in actual field work, and to do so in cooperation with extension services, project administration and planning agencies. This would make sure that research is directed to problems whose solutions are of direct importance to agricultural development. The effectiveness of such programmes would be further enhanced by the participation of research and training organizations from countries with highly developed water management systems.

Action at the national, regional and interregional levels is needed to coordinate the field programmes and standardize methodologies, to collect and disseminate information, to organize the training of staff and to provide financial support. It is considered that water management centres at this level, functioning as focal points for these tasks rather than as research institutes, are urgently needed.

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Paper 4.03

PROGRESS AND PROBLEMS IN DRAINAGE DESIGN

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Summary

An adequate body of theory is available to predict the behaviour of drainage systems. This theory, however, has not been packaged in a manner that makes its application as convenient or pertinent as it could be, in part because of a lack of data relating a certain degree of drainage to economic returns.

For rainfed areas, the effectiveness of drainage probably should be evaluated in terms of the soil water content of the rootzone and/or the surface soil, rather than the height of the water table. For irrigated areas, the primary concern is removal of excessive salts. In the latter case, the interaction between irrigation management (and cropping pattern) and drainage requirements must be recognized. Furthermore, in both cases, the drainage facilities must be designed with due regard for the natural drainage rate.

Finally, the impact of drainage is not restricted to the farm; in some cases, the off-site effects are important and may even dominate.

Introduction

Since the pioneering work of Hooghoudt some forty years ago, and that of his predecessors, a tremendous body of new drainage theory has become available. We now have a host of equations that describe, with various reasonable assumptions, the flow of water through soils and to drains. Notwithstanding this body of knowledge, practical drainage design more often than not is still based on guides derived from experience rather than on analytical formulations. The step from theory to practice still is a formidable barrier.

Here I shall make a few remarks about analytical solutions of transient watertable problems; consider some aspects of unsaturated flow; briefly address drainage design criteria; and, finally, call attention to certain aspects of water quality.

Analytical solutions

Whereas the usefulness of steady-state analyses is readily granted, the situations most commonly encountered in the field are better described by non-steady formulations. The best known and most frequently used solution for the typical problem of a falling watertable over parallel drains is based on a linearized version of the differential equation resulting from the Dupuit-Forchheimer assumptions,

$$K \frac{\partial}{\partial x} \left(h \frac{\partial h}{\partial x}\right) + R = f \frac{\partial h}{\partial t} \tag{1}$$

where K represents the saturated hydraulic conductivity; f an assumed constant drainable pore space, R precipitation and h the hydraulic head referred to the impervious layer; x is used to represent the horizontal coordinate and t is time. Linearization yields, with $\bar{h}$ an assumed constant depth of the water-bearing stratum,

$$K \bar{h} \frac{\partial^2 h}{\partial x^2} + R = f \frac{\partial h}{\partial t} \tag{2}$$

The resulting Fourier series solution for R = 0 is often referred to in the USA as the Glover equation (Dumm, 1954). If a correction is made for convergence near the drain by means of Hooghoudt's equivalent depth (van Schilfgaarde, 1963; Moody, 1966), the results obtained are entirely adequate as long as the flow above the watertable can be ignored and the equation is used for a single drawdown event. Tapp and Moody (Dumm, 1964) later modified the Glover equation somewhat to the solution now used routinely by the U.S. Bureau of Reclamation by using a 4th degree parabola initial watertable in place of a horizontal watertable, as originally assumed. However, the same problem has been solved without linearization, i.e., without the assumption that the depth of the water bearing stratum stays constant during drawdown. This writer (1964) presented the solution

$$t = \frac{fS^2}{9KD_e} \ln \frac{m_o(2d_e + m)}{m(2d_e + m_o)} \qquad (3)$$

where t is the time required for a mid-spacing drop in watertable from m_o to m above the drain axis, d_e the equivalent depth below the drain axis, and S the drain spacing. Equation (3) reduces, in the limit as $d \to 0$, to the Boussinesq equation,

$$t = \frac{2fS^2}{9K} \frac{m_o - m}{m_o m} \qquad (4)$$

as it should (Raats and van Schilfgaarde, 1974). The initial condition implied in both Eqs. (3) and (4) is an elliptically shaped watertable. Thus a simple-to-use solution has been at hand for some time that avoids the restriction introduced in the Glover equation through linearization, and the substantial errors that result for relatively large drawdowns.

The most useful application of falling watertable theory, however, considers a sequence of recharge events. Werner (1957) and Maasland (1959, 1963) applied the principle of superposition to intermittent but regularly periodic recharge events as may be visualized from irrigation. Krayenhoff van de Leur (1958) and van Schilfgaarde (1965) considered the more general case of arbitrarily distributed precipitation. Because the principle of superposition requires linearity, Eqs.(3) and (4) above, cannot be used for this purpose; the authors cited used an appropriate, if mutually different, starting point in each case. Similarly, direct use of the modified Glover equation (Dumm and Winger, 1964) introduces a systematic error, resulting in overconservative spacing recommendations, because of the forcing of the initial condition for each recharge impulse. McWhorter (1977) discovered this problem and represented a corrected solution.

The U.S. Bureau of Reclamation routinely uses the concept of periodic yearly recharge patterns from irrigation in its drainage design. As just pointed out, one can find fault with the detail of the procedure, but the principle is sound. A key assumption concerns the amount of deep percolation following each irrigation. In a recent study by the U.S. Bureau of Reclamation (unpublished), it was demonstrated by a linear programming technique that the farmers' choice of cropping pattern could have as large or larger effect on the annual drainage volume from an irrigation district as the irrigation management for a fixed cropping pattern.

In rainfed agriculture, drainage design based on the probability distribution of rainfall and consequent deep percolation seems not to be practiced. A number of research studies (e.g. Vaigneur and Johnson, 1966; Wiser et al., 1974; Skaggs, 1977) have demonstrated the potential, but application by action agencies or consulting engineers has not been encountered. We shall return to this point later.

Flow above the watertable

All of the equations and techniques discussed so far consider only saturated flow bounded by a watertable. This clearly imposes a major limitation. There have been numerous studies in which the total flow system was considered, while in others an intermediate approximation was used. As to the latter, the simplest device still is to lower the effective impervious layer by an amount equal to the thickness of the capillary fringe, whatever way one wishes to define it. With respect to the former, most workers have chosen to resort to numerical solutions of Richards' (1931) equation for unsaturated flow. Skaggs and Tang (1976), for example, analytically determined the position of the watertable as a function of time for a range of geometries and compared the results based on the Richards' equation with those obtained from Eq.(1), with and without a convergence correction, and with and without a capillary fringe; they derived a (variable) drainable pore space from the steady-state moisture characteristic. McWhorter and Duke (1976), on the other hand, chose to adapt the Glover equation by making a fairly involved correction for flow above the watertable and water storage in that zone, while retaining in part the non-linearity of the differential equation. In part, this was accomplished by replacing the factor $K\bar{h}/f$ in Eq. (2) by an analytically derived expression that takes account of the volumetric rate of drainage from the region between the soil surface and the watertable as the watertable drops. Their procedure, as expected, results in a substantially faster rate of watertable drop than does use of the Glover equation. Even if the Glover equation is corrected by increasing the thickness of the aquifer by the estimated thickness of the capillary fringe, it still results in a somewhat slower rate of drop than determined from more rigorous procedures. This and similar studies are significant in that they provide an estimate of the magnitude of error introduced by the simpler analytical tools that ignore the flow above the watertable. They have a serious limitation, however, in that in principle they do not lend themselves to superposition. A further, and possibly more severe, limitation is that the soil parameters required are expensive to determine and may vary widely over short distances. On the other hand, since probabilistic - or dynamic equilibrium - evaluations generally will call for computer computation in any case, it is certainly possible to develop families of tabulated or plotted solutions for classes of situations that take into account all pertinent soil hydrologic variables.

Drainage criteria

From the foregoing brief discussion, I conclude that we have adequate analytical tools to describe the behavior of the watertable, or even the time course of the water content in the rootzone. Lacking is a sufficient data base to interpret such calculations in terms of the economic return from a drainage system. Wiser et al. (1974), for example, calculated the frequency of flooding for prescribed periods and related these results to published yield responses of alfalfa to make an economic analysis. As they pointed out, they had no basis for assessing damage due to watertable rises that did not reach the surface. Young and Ligon (1972) calculated how long watertables were expected to be above certain levels for given recurrence intervals, drain spacings and hydraulic conductivities at a location in South Carolina. They also related watertable height directly to soil water content. They left to the reader, however, the interpretation of these data in terms of crop response.

Possibly we tend to forget that the watertable, a convenient criterion for purposes of measurement and calculation, has no particular significance when it comes to plant growth. Duke (1973), among others, explicitly proposed that drainage design be based on adequate aeration. He used the relationships between relative saturation, matric potential and hydraulic conductivity proposed by Brooks and Corey (1964), and assumed that the equivalent depth of a capillary fringe, w, as calculated (Myers and van Bavel, 1963) from

$$w = (1/K) \int K(z)dz \tag{5}$$

integrated from the watertable to the soil surface, could be used to estimate the zone in which gaseous diffusion was insufficient to maintain plant roots. The main point is simply that attempts have been made repeatedly to provide better indices for the effectiveness of a drainage system, but to my knowledge, these concepts have not yet been packaged to be of direct use to practising designers.

Under irrigated conditions, an important criterion for drainage design is the predicted salinity in the soil solution. It has been customary to express this criterion in terms of a leaching requirement (LR) which is derived by imposing on the leaching fraction (LF) the restriction that the soil solution leaving the rootzone cannot exceed a prescribed value. The LF simply states that, at steady-state, the mass of salts removed from the rootzone through drainage equals that brought in with the irrigation water (USSL Staff, 1954):

$$LF = V_d/V_i = C_i/C_d \tag{6}$$

Here V and C stand for volume and concentration, and the subscripts i and d for irrigation and drainage water. The LR, then, becomes

$$LR = V_d^*/V_i^* = C_i/C_d^* \tag{7}$$

where the asterisks distinguish the desired, or required, conditions from those actually encountered. To determine numerical values for V_d^*, the USSL used to advocate somewhat arbitrarily that C_d^* could be taken equal to the concentration of the saturated soil extract at which a 50 per cent reduction in crop yield was obtained in field experiments with artificially salinized waters that utilized high leaching fractions. Such experiments tend to result in uniform salinity throughout the rootzone.

More recently van Schilfgaarde et al. (1974) proposed that, although these recommendations were safe, they led to unnecessarily high LR's. Briefly, the reasoning is as follows: Under quasi-steady-state conditions, salinity profiles are not uniform; they tend to take on an S-shape, with the salinity near the surface about equal to that of the irrigation water, but increasing asymptotically to a maximum at the bottom of the rootzone. The plant root system is able to extract water from the soil solution with minimal ill effects until its concentration reaches a maximum peculiar to the crop. This maximum can be approximated from existing data on crop tolerance to salinity by extrapolation to 100 per cent yield reduction. Maas and Hoffman (1977) recently summarized many of the existing data in a table of coefficients A and B for the equation

$$Y = 100 - B(S_e - A) \qquad (8)$$

where Y represents relative crop yields and S_e the electrical conductivity of the saturation extract in mmho/cm (1 mmho/cm = 0.1 S/m). Thus for Y = 0,

$$S_e^* = A + 100/B \qquad (9)$$

and, with appropriate units, the value in Eq.(7) for C_d^* (with concentrations expressed in electrical conductivity) would be S_e^* adjusted from saturation extract to field water content (S_w^*). In the absence of specific data, this correction may be taken as

$$S_w^* = 2\ S_e^* \qquad (10)$$

Application of this concept will result in a reduction in LR from earlier recommendations by a factor of 3 to 4. It should be stressed that this concept is based on reasoning supported by limited data and requires further verification before it can be advocated with confidence. There is no question, however, that LR's can be reduced below those generally advocated.

A change in LR should translate directly into a change in drainage requirements. However, a number of other variables enter in before a drainage criterion can be established for design purposes. Among them is the need to distinguish between natural drainage rate and the additional drainage required through a man-made drainage facility - frequently a difficult task, especially in new lands to be developed for irrigation. Also important is the effect, mentioned before, of a change in cropping pattern. Here we call special attention, however, to the close interrelation between irrigation management and drainage need. Even if the above postulate on LR is proven fully justified, one must take account of both special and temporal variability, not to mention the farm operator. With infrequent irrigation, both the matric and osmotic components of water potential will fluctuate significantly during an irrigation cycle and the steady-state concepts outlined must be adjusted accordingly. With most irrigation systems, the areal uniformity of water intake deviates substantially from the ideal, and a leaching fraction that is barely adequate on the average will be inadequate on some part of the field. Thus drainage design must be seen as an integral part of the development of a total water management plan.

It does not follow that one should ignore the possibility of more efficient irrigation practices and consequent reduced drainage requirements. On the contrary, there are often good reasons, including the conservation of water and energy resources and the savings in costs, to design and operate systems to take full advantage of the potential.

Environmental considerations

Agricultural water management, while primarily concerned with on-farm conditions and crop production, clearly impacts the environment off-site. It must be viewed as an important component of total natural resource management. Only a few observations will be made here that relate directly to the subject matter of this conference.

The implications of the above discussion of LR and irrigation practices are often more significant in terms of downstream effects than for the farm operator. Extensive studies in the Colorado River Basin, for example, have shown that the most cost effective approach to maintaining or improving water quality in terms of salinity downstream starts with the improvement of on-farm water management. Two processes are especially important. When the leaching fraction is reduced, independent of irrigation water quality, there is a shift from soil mineral dissolution to salt precipitation. Thus changes in drainage rates (i.e., irrigation practices) will affect the total amount of salt in solution that must be managed. Secondly, in some instances drainage waters displace saline groundwaters, thus adding disproportionally to the river's salt burden. Alternative choices for amelioration, such as desalting or disposal of drainage waters, carry heavy costs. Another type of situation is illustrated by the Central Valley in California. Since drainage water disposal out of this mountain-ringed basin would involve a tremendous capital investment even if it were socially acceptable, any method that reduces drainage volumes substantially without endangering crop production would pay high dividends; in this instance, evapotranspiration by plants is a viable alternative to exporting for disposing of a substantial part of the waste water.

Two brief illustrations put these considerations in perspective. The Wellton-Mohawk Irrigation and Drainage District in Arizona historically receives about 640×10^6 m^3/yr of irrigation water, delivered in open canals through a pump lift of about 55 m. This water is used to irrigate about 26,000 ha and results in some 270×10^6 m^3/yr of drainage water pumped from nearly 100 wells at an average salt concentration of above 3,000 mg/l. To meet an agreement with Mexico relative to the quality of water delivered, there were two extreme options:a desalting plant requiring 370×10^6 kwhr/yr that would spill about 50×10^6 m^3/yr of brine to the ocean, blending the remaining waters to the agreed concentration; or increased irrigation efficiency to technically feasible but extremely difficult to obtain levels, also spilling about 50×10^6 m^3/yr (one-fifth the current drainage volume) to the ocean. The implications are clear. The final solution is expected to be a combination of the two extremes.

In the Central Valley, we have proposed that drainage return flows be separated from irrigation water supplies and, in a concentration range of 0.5-0.9 S/m, be reused to irrigated tolerant crops. We speculate that after this use, the new drainage waters might well still be used to produce biomass from halophytes that can be used to produce methane or other industrial products. The first step alone would substantially reduce the disposal problem. Alternatives include reductions in irrigated areas, selling the drainage water for industrial use such as cooling towers; or constructing a massive central drain outlet. This proposal should open up a wider range of options, and probably more acceptable ones, than the more direct implementation of greatly improved irrigation efficiencies.

Finally, there is an area of concern where drainage engineers and soil physicists can provide a useful service. As regulatory control over water resources increases, it becomes more important to show definitive relations between land and water management practices and drainage water quality. Failure to establish such relationships may well result in unwarranted restrictions. This challenge requires, as a starting point, that attention be given to flow paths and travel times, and not just potential distributions and gross discharge rates. Existing potential flow theory, in principle, can provide the needed answers. However, the otherwise useful approximations, such as the Dupuit-Forchheimer approach, cannot be used. Jury (1975) made a start in this direction and Raats (1977) expanded on this problem analysis.

Epilogue

Not much will be gained from further refinement of existing drainage theory or from development of new solutions to abstractly posed problems. The challenge ahead is to imaginatively apply the existing catalogue of tricks to the development of design procedures that are convenient and readily adapted by practising engineers. This calls for better definition of drainage design criteria and, no doubt, expansion of the data base for crop response as well as trafficability. It will require expression of such criteria in terms of recurrence intervals. This, in turn, will be facilitated by the increasingly greater availability and capacity of computers. Thus the advantages van Schilfgaarde and others claimed in the past for closed analytical solutions have been (partly) dissipated.

Closely related is the need to consider drainage problems as part of a total water management scheme. The recent drought has emphasized that need in The Netherlands; water quality concerns have driven it home in the Western USA. Challenging opportunities await us in devising means to better use our dwindling natural resources worldwide. Drainage specialists will play an important role in meeting these challenges, but only if they can put drainage in the proper perspective.

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Paper 4.04

CONSTRUCTION AND MAINTENANCE TECHNIQUES FOR SUBSURFACE PIPE DRAINAGE SYSTEMS IN IRRIGATED LANDS

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Summary

Installation of subsurface drains in irrigated lands requires specialized trenching machines capable of excavating a suitable trench to depths of at least 3.7 meters. Trenching machines should also be capable of installing a designed, well-graded gravel envelope which will provide adequate permeability for convergence, prevent fines in the base material from moving into the envelope and drain line and provide the required bedding support for corrugated plastic drain tubing.One of the more complex problems in drainage of irrigated lands is establishing a stable drain trench when saturated, unstable soils are encountered. Stabilization of drain trenches can be accomplished by using a coarse gravel bedding, installing a system of well points, or installing a temporary dewatering drain below the grade of the permanent drain. Backfilling the trench after the drain tubing has been installed must follow some type of backfilling technique that will insure consolidated backfill to the original ground surface. After the drain has been completed the effectiveness and quality of installation should be evaluated by flushing or pulling a torpedoshaped plug having a diameter 1-inch less than the drain diameter through the drain in a manner that will not affect the installation or efficient functioning of the drain.A TV camera can be used after the drain is in place to observe obstructions, determine tubing deflection, and condition of the water inlet perforations. Periodic inspection and maintenance are essential for keeping the drains from plugging with roots and from bacteria. Cleaning equipment consists of high pressure jetting equipment and high speed auger type cutting heads. Copper sulfate can be used to kill roots and sulfur dioxide gas or safe sulfuric acid in pellet form can be used to reduce iron bacteria.

Introduction

Agricultural drains have been installed before man's recorded history (Maierhofer, 1967). Some ancient systems were simple, some were elaborate, but few were entirely successful. Part of man's trouble with drainage systems has been due to lack of understanding of the physical and technical problems involved in designing and constructing the systems, and part has been due to neglect of the completed system. Drainage is not yet an exact system science and probably never will be, but as man learns more about the complicated plant-soil-water relationships and develops better subsurface drain design, construction, and maintenance techniques, drainage systems will become more economical, successful, and permanent.

A. CONSTRUCTION TECHNIQUES

Most of the existing literature on subsurface drainage covers the investigation, location, and design of these drainage systems, but there is only limited information about construction and maintenance techniques. After the designer is satisfied with the design of a drainage system, the design and construction requirements are turned over to the drainage contractor who has the responsibility of installing drains that control the watertable as the designer planned, have the minimum maintenance problems and last forever. Few people watching a smoothly operated drain installation realize that the drainage contractor must be a highly skilled, experienced person, who through his own mistakes or the mistakes of others has learned how to handle all types of problems from dealing with an irate farmer to installing drains in unstable material that perform as designed. As any drainage contractor knows, there will be a new problem as well as old problems every day so he has learned to "hope for the best, but expect the worst" each morning when he starts his equipment.

Equipment

Installation of subsurface drains in irrigated lands requires specialized trenching machines capable of digging to depths of at least 3.66 meters (12 feet). For drainage of irrigated lands economic drain depth ranges from 2.1 to 3.0 meters (7 to 10 feet; Christopher and Winger, 1975), so the designers will try to keep the average depth within this range. Trenchers that dig only to 2 meters (6.5 feet) cannot install satisfactory drains without removing the overburden soil down to within 2 meters of the bottom grade of the trench. For areas of high watertable, this amount of scalping will place the machine on unstable material and it will not be able to operate. Therefore, only the larger machines especially designed for deep drains are suitable for installing drains in irrigated lands. For general drain installation, where all types of soils and cementation problems are encountered, the so-called conventional trenchers will do satisfactory work. These consist of the ladder- and wheel-type trenchers capable of maximum sustained speeds in uncemented materials at depths of about 2.5 meters (8 feet) of about 122 to 150 meters per hour (400 to 500 feet per hour). For medium-to-heavily cemented materials the large wheel-type trencher performs the best. The teeth and sometimes the entire bucket on the ladder-type trencher will break off when boulders or a section of heavily cemented material are encountered. Both the ladder- and wheel-type trenchers will function fairly well in unstable soil as long as the surface will support the weight of the trencher and the trencher can be kept in motion. If either type of trencher stops in unstable material the unstable material caving against the shield can cause the drain pipe or tubing to be pushed out of alinement and grade. Prices for the larger type wheel- and ladder-type trenchers with a laser grade control system, shield, and gravel hopper but not continuous backfilling equipment, are in the range of $175,000 to $225,000.

The larger chain-type trenchers are capable of installation speeds of 180 to 250 meters per hour (600 to 800 feet per hour) at depths of about 2.5 meters (8 feet) in uncemented, friable material. However, any large rocks or cemented material will damage the fastmoving teeth and in most cases will break the chain. Also in unstable materials the chain-type trencher churns the supersaturated material to a consistency so that the digging teeth will not remove the material from the trench, and grade and alinement requirement cannot be maintained. Many contractors who install thousands of meters of plastic drain tubing annually in all types of material are equipped with both the large wheel-type and the large chain-type trenchers, and each machine is used where it functions the best and most economically. Some drain contractors build their own trenchers, usually starting from one of the commercial-type trenchers and adding the features they believe will assist them in installing better and more economical drains. This is an expensive project, but the contractor can sometimes develop a trencher that does function better than the commercial trenchers. However, there are probably more inventor contractors that go out of business because of debts than those that succeed.

There are a large number of plow-type machines that are operating throughout the world. These plow-type machines install the drain tubing without the necessity of excavating the material from the trench. Many are capable of high speeds, and can install pipe up to 2 meters (6.5 feet) deep. This type of machine performs satisfactorily only in the more sandy, lighter textured soils, free of rocks and cemented materials. In soils high in silt and clay with high moisture content the plow shoe will form an almost impermeable seal underneath the drain tubing. This type of machine installs an economical drainage system per meter of drain line but on a hectare basis the completed drainage system will cost about the same as conventional trenching machine installation, because the more shallow installation depth results in additional tubing and envelope material.

All drain-installing equipment must have a method of placing some type of envelope material around the drain. The designer will specify the type of envelope material required and the contractor must equip his machine to place the material. There is some disagreement on what materials should be used to surround the pipe or tubing, but research studies indicate that a designed, well-graded gravel envelope produces the most water, prevents fines in the base material from moving into the envelope and drain line and provides the required support for corrugated plastic drain tubing (Winger and Ryan, 1970). Synthetic envelope materials are being used where the source of gravel is limited and expensive. They are appropriately called "filter materials" because that is their sole function. When used in sand and gravel base materials with little or no silt or clay, the synthetic materials are satisfactory. However, when used in soils containing silts and clays they eventually develop a "filter-cake" which in time prevents the ground water from entering the pipe or tubing. Also, when synthetic materials are specified around plastic tubing the trenching machine should be equipped with buckets only a few inches wider than the outside diameter of the tubing and a shoe to form a properly shaped bed for the tubing so it will have adequate support

When gravel envelopes are specified, the trencher must be equipped with a shield which includes a gravel hopper that is capable of providing a continuous flow of gravel around the tubing. For unstable soils that flow upward when the overburden pressure is removed, the gravel must be placed completely around the tubing or pipe before it leaves the protection of the shield. For all subsurface drains where a gravel envelope is used, the trenching equipment and the method employed in excavating the trench, laying the drain pipe and placing the gravel envelope material should be such that the in-place gravel is in contact with undisturbed soil at the sides and bottom of the trench.

When excavation at the bottom of the trench is in very unstable material,the trenching equipment should be capable of excavating to additional depths to provide for installing stabilizing material. The material used for stabilization should be hard, dense, and durable rock of sufficient size and gradation that it will establish a bedding for the gravel envelope material and drain pipe. Since the material cannot be run through the gravel hopper because of its size, it must be placed in the trench by a backhoe or frontend loader.

Besides the trenching and laying equipment, the contractor should have a backhoe or drag line to excavate and place the outlet pipe, excavate the trench when large boulders are encountered; install the pipe or tubing at road, lateral, and canal crossings; and set manholes. When the gravel is placed at convenient locations along the drain line the contractor will also need a front-end loader to move the gravel from the storage site to the machine. He can have his gravel hauled from the gravel pit to the site and dumped at convenient locations, or he can build special gravel trucks that use auger-type equipment to place the gravel directly into the gravel hopper. This procedure saves gravel, keeps out unwanted clay and silt, and permits the machine to continue without stopping to have the hopper filled. In unstable, swampy areas half-track trucks work very well for keeping the gravel hopper filled.

The drainage contractor should also have a bulldozer to backfill the trench, to scalp when required, and pull the trencher through unstable areas. A pick-up or extra truck is needed to distribute pipe and tubing, haul equipment, and move the crews.

For more convenient grade control equipment a number of companies now manufacture laser control systems. These systems eliminate target setting or changing and permit getting the machine in the ground much faster than when setting targets. Also, when all equipment is working properly, and the laser system is controlling the grade, there are few mistakes in grade. However, the cost of a good laser control system ranges from $14,500 to $15,000 usually only the full-time drain contractors find the equipment profitable.

Other smaller pieces of equipment that are essential for subsurface drainage pipe installation include a good water and sand pump to control the water in the drain when constructing manholes and supplying water for puddling the trench excavation; a transit, level, and rod for any surveying work required; drain testing equipment such as sewer balls and wooden torpedo-shaped plugs which are floated or pulled through the pipe to be sure it is operating satisfactorily; water tanks used for flushing drains when filled with mud; and a welding outfit for field repairs.

Construction requirements

Drain construction, like all other construction work, requires a clear understanding of what must be done, what materials will be used; what the finished product will look like, how the finished product will perform, what inspection will be made during the construction period, what inspections and tests are required for acceptance of the drain, and how contract adjustments will be made (USDI Bureau of Reclamation, 1976).

This paper will not attempt to cover all construction performance requirements for a successful, permanent irrigated lands drainage system, but only a few of the requirements that can mean the difference between a good drain and one that will not function at all or only for a short time.

One of the more complex problems in drainage of irrigated lands is stabilizing drain trenches when unstable soils are encountered. These conditions usually are found where the drain must be constructed through fine sands and high silt base materials with high ground water, but occasionally are encountered in heavier textured soils under pressure. When unstable soil conditions exist due to high ground water and the contractor cannot install a satisfactory drain even by maintaining a steady installation speed through the area, he may be required to overexcavate the bottom and sides of the trench and backfill with a coarse gravel. Unwatering of the trenches, where coarse gravel backfill is required to placed, should be to such an extent to ensure that the stabilizing gravel is deposited in such a manner as to provide a satisfactory foundation for laying the drain pipe. The coarse gravel used for stabilizing should be free of vegetable matter, bentonitic clays, and other deleterious substances. Because of the smaller gradation requirements, a designed, well-graded envelope material is not suitable for most stabilization requirements. A good gradation for a stabilizing gravel is as follows:

Retained on the	(5-inch screen)	None
Retained on the	(4-inch screen)	0% to 20%
Retained on the	(3-inch screen)	0% to 30%
Retained on the	(2-inch screen)	20% to 50%
Retained on the	(3/4-inch screen)	20% to 50%
Passing	(No. 4 screen)	Less than 5%

There are other ways of stabilizing the soil along the centerline of the drain besides the use of coarse gravel. These include installing an extensive system of well points and draining the immediate vicinity of the drain by pumping. However, the hydraulic conductivity of the soil must be sufficient to create the desired drawdown effects in a reasonable length of time. This is an expensive method and used only when the conditions are suitable and all other methods have failed.

Another method of stabilizing the soil in the vicinity of the drain centerline is to install a 76.2- to 101.6-millimeter (3- to 4-inch) diameter plastic drain tubing, surrounded by a gravel envelope, below the bottom grade of the permanent drainpipe and run the effluent into a sump where it can be pumped into suitable disposal channels. Since the temporary drain will be abandoned as soon as the permanent pipe has been installed, less expensive tubing can be used and less rigid requirements as to grade and alinement can be permitted. This type of stabilization requires trenching equipment capable of digging to depths of 3.7 meters (12 feet) or better which results in an expensive installation cost. No matter what method of stabilization is used, drains properly installed in unstable material are expensive.

Pipe outlet structures

The construction of suitable outlet structures for subsurface pipe drains is essential in reducing the drainage maintenance cost and providing a better operating drainage system. Many types of pipe material have been tried for the outlet system, but only the bituminous-coated or asbestos-bonded galvanized corrugated-metal iron or steel pipe have proven satisfactory over periods of 30 to 40 years. The corrugated metal pipe should have a diameter slightly larger than the drainpipe so it can extend at least 0.3 meters (12 inches) back over the drainpipe. Depending upon the grade conditions of the open drain, the corrugated metal pipe should be at least 0.3 meters (12 inches) above the normal water surface in the open drain and be long enough to extend past the side slope so the drain effluent, even for very low flows, will have free fall into the water in the open drain. Both the sides and bottom of the open drain should be protected with a coarse gravel blanket or placed riprap for at least 0.75 meter (2.5 feet) upstream and downstream from the pipe. This will not only prevent erosion caused by the falling drain effluent, but will provide a section of stable channel so the outlet can function as designed over longer periods of time. Any damage during installation or cleaning operations to the galvanized coating or bituminous covering should be repaired by application of an approved compound. Any unit that is damaged beyond repair in hauling, handling, or otherwise should not be installed.

Backfill techniques

Backfill in drain trenches is usually accomplished by pushing the excavated material back into the trench and mounding the excess material over the trench with little or no compaction. As soon as the contractor moves off the land, the farmer immediately removes the mound and gets the land ready for planting. After one or two irrigations the material in the trench settles and the farmer has problems in cultivating, ditching, spraying, and irrigating. With sprinkler irrigation, gullies may be formed along the drain trench settlement causing the sprinkler wheels to stop moving. This situation results in damage to the sprinkler equipment, young plants are washed out, roads are flooded, and more mature crops are damaged from too much water.

To alleviate this problem, a backfilling technique has been developed to consolidate the backfill in drainpipe trenches through cultivated fields and croplands by puddling (Frogge and Sanders, 1977). As nearly concurrent with placing the gravel envelope and laying the pipe as practicable, the contractor backfills the trench to a minimum height of 0.6 meter (2 feet) over the pipe. This keeps the plastic tubing or concrete pipe protected during the rest of the backfill procedure. No rocks larger than 76 millimeter (3 inches) should be permitted within 0.30 meter (1 foot) above the pipe. As soon as practicable the contractor should continue the backfill operation to a height of approximately 0.30 meter (1 foot) below the natural gravel surface which provides a holding ditch for the puddling water. Earth dikes are then constructed across the ditch at 200-foot intervals to provide suitable conditions for puddling. No special compacting is required, but the bottom of the ditch should be consolidated so there are no channels directly to the drainpipe which could cause piping of backfill into the envelope material.

Puddling is accomplished by pumping water, from any source available, into the prepared ditch. The amount of water pumped should not exceed about 15 cubic meters (4000 gallons) per 30.5 meters (100 feet). Upon completion of the filling operation all of the ponded water should be allowed to seep into the backfill material. No ponded water should be removed by surface drainage. All material in the trench, including material in and below the ditch, should be completely saturated by the ponding process. During the ponding operations the drain contractor should maintain a close observation of the work and promptly fill any sinkholes or areas of excessive settlement. After all the water is out of the trench, the contractor can complete backfilling the trench to the elevation of the adjacent natural ground surface. The final backfilling need not be consolidated, but equipment should be routed so as to distribute the compacting effect to the best practicable advantage. When the trench has been puddled for settlement no mounding should be permitted over the trench because the entire process was developed to leave the surface of the field in the same condition as before drainage. After completion of backfilling a good practice is to have the contractor scarify all areas disturbed by construction of the drains.

Stretching of corrugated plastic drain tubing

One other serious problem encountered during the installation of corrugated plastic drain tubing is the reduction of strength due to excessive stretching during the installation process. Using the parallel plate pipe stiffness test, a minimum allowable "pipe stiffness" has been established for a 5 and 10 per cent deflection. Corrugated plastic drain tubing produced by the leading tubing manufacturers meets or exceeds the specifications requirements with zero stretch. However, laboratory studies indicate that even the best tubing retains only about 90 per cent of its original stiffness with 5 per cent stretch both for the 5 and 10 per cent deflection tests. At 10 per cent stretch the tubing shows a retention of only about 80 per cent of its original stiffness and at 15 per cent stretch the tubing retains only about 70 to 75 per cent of its original stiffness. At 20 per cent stretch the tubing retains less than 65 per cent of its original stiffness. These studies indicate that tubing must exceed the minimum pipe stiffness by about [illegible] per cent when the specifications have a maximum 5 per cent stretch.

Most stretch occurs between the keeper sprockets, which guide the tubing into the shield, and just after the tubing has left the shield. If the sprockets do not allow the tubing to feed over them freely they catch the tubing and cause the pipe to stretch between the sprockets and where it is anchored in the trench. When this happens the total stretch for 30.5 metres (100 feet) might be within the specifications' allowance of 5 per cent, but the 1.5 metre (5-foot) stretch probably occurred in a 3.0-metre (10-foot) length which is a 50 per cent stretch and pipe failure will result. Therefore, it is essential during the installation of corrugated plastic drain tubing to have free-running keeper sprockets that cannot catch and hold the tubing during normal operating speeds of the trenching machine. Only an alert inspector or machine operator can make sure there will be no failures because of excessive stretch.

Evaluation of construction techniques

Once a subsurface drainage system has been installed and backfilled, it is seldom looked at until lack of adequate maintenance results in serious trafficability problems and reduced crop production. Most subsurface pipe drains develop problems right after construction or when roots start growing into the slots a few years after construction. To determine the conditions of the drain after the initial backfilling, but before puddling, a sewer ball or torpedo-shaped plug having a diametre 1 inch or less than the inside diametre of the drainpipe should be flushed or pulled through the pipeline in a manner that will not damage the pipe, affect the gravel envelope, or otherwise decrease the efficient functioning of the drain. The plug or ball should be nondeformable and only a reasonable pulling force should be used when pulling the ball or plug through the pipe. Reasonable pull in this case means a pull by one man using about 11.3 kilograms (25 pounds) pull as measured on a spring scale. Any obstructions resulting from material in the pipe, pulled joints, or collapsed plastic tubing which causes the floating ball to stop or the need for a greater force on the pulled plug than permitted will justify the need for digging up the pipe and replacing it with new pipe or tubing.

TV-camera inspection

As long as the drain is relatively clean, reoccurring obstruction problems and growth of roots or bacteria sludge in the slots or holes of the corrugated drain tubing can be inspected with a special TV-camera. The troubled areas can be observed on the monitoring screen and video tapes can be made for more detailed studies. If polaroid pictures are desired, more clearly defined images can be obtained by taking the pictures directly off the monitoring screen during the inspection rather than the video tape.

There are two types of TV-camera heads that can be used. The regular borehole camera takes pictures directly ahead and is used for observing obstructions, determining tubing deflection, observing errors in grade, and evaluating the general appearance of the tubing. The revolving head TV-camera can be revolved to a 90° angle for observing the condition of the slots or holes and build up of concentration of iron bacteria sludge on the sides of the tubing. Specially designed deflection indicators can be installed on the borehole TV-camera sled ahead of the camera so a continuous observation and video tape can be obtained of the deflection in the plastic tubing. This type of study is especially important where the stabilization of the trench bed is difficult and the conformance to construction specifications is uncertain.

B. MAINTENANCE OF SUBSURFACE DRAINS

Introduction

Efficient and well-maintained subsurface drainage systems will effectively control the watertable in irrigated lands long after the storage and irrigation supply systems cease to function. Timely performance of preventive and regular maintenance is absolutely necessary if the systems are to perform the functions for which they are designed and installed (Winger, 1973). A buried drain very soon becomes "out-of-sight and out-of-mind", so definite inspection and cleaning schedules must be initiated and followed. Many of the large irrigation projects have thousands of kilometres of subsurface drains, so even a periodic inspection program is expensive. Seldom is there enough money budgeted annually for the maintenance requirements even when the periodic inspection indicates the repairs are needed to keep the drains operating effectively.

Subsurface drainpipe

Pipe drainage systems, properly installed, generally require only minimum care to keep them operating satisfactorily. Newly constructed systems require close vigilance during the first year or two of operation, but after the trench has stabilized and most subsidence has taken place only periodic inspections are required. Proper care of the system during the first year or two will increase the efficiency of the drains and many times eliminate the need for expensive repairs in later years. Failures or partial failures in subsurface pipe drainage systems usually are associated with unstable soil conditions during construction which causes shifts in pipe alinement and grade, pulled joints, plugged water inlet joints or slots, buried or plugged drain outlets, collapsed tubing, and plugged manholes. A pipe drain requiring attention will be found during periodic inspections by noticing seeped or soft areas over the drain, sink holes over the drain, manholes full of fine material, water backing up in the manholes, trees or especially high weeds growing over the drain, iron sludge in the manholes or at the outlet, and an angry farmer trying to get his truck or tractor out of a soft area.

When practical, a flow record should be established at each manhole and at the pipe outlet after the drain has stabilized. If the discharge at any of the measuring points drops suddenly with continued irrigations there is a good possibility a segment of the drain is plugged or partially plugged and measures should be taken to locate the plugged spot.

After initial stabilizing period, tree and plant roots are the principal cause of plugging in subsurface drains. Periodic treatment with copper sulfate will kill the roots which can then be cut off and removed by the use of a high-speed auger-type cutting head equipped with a number of sharp teeth or by the use of a high-pressure jet. The cutting or jetting equipment is always started from the downgrade side of the root problem area, so the cut roots can float or be pulled out to the manhole and be removed. Experience has shown that annual applications of about 0.9 kilogram (2 pounds) per 305 metres (1000 feet) of copper sulfate crystals introduced into the manholes in bags during periods of low flow will prevent most roots from growing into the drain. When roots have clogged the drain this amount should be doubled and repeated until the roots have been killed. The use of small crystals measuring about 12.7 millimetres (1/2 inch) is preferred because when in contact with the root mass they dissolve copper solution over a longer time period than the finer grade material. When the flow in the drain is completely stopped by roots and microscopic organisms that grow in plugged drains, some flow must be restored by jetting or mechanical means in order that the copper sulfate crystals may be dissolved and the resultant solution carried through the root masses.

Occasionally dead fish are observed in the open drain below the treated subsurface pipe drain outlet. Unless excessive quantitities of copper sulfate were used, these fish probably died not due to any toxic effect of the copper sulfate, but rather to clogging of the gills by dead organisms or reduction of oxygen by the organic matter of the killed organisms. There is considerable disagreement between maintenance personnel and fish biologists as to the amount of copper sulfate that can be safely used. Fingerling trout appear to be the most susceptible to copper sulfate with claims that dosage as low as 0.14 milligram per liter (1.2 pounds per million gallons) have caused the young trout to die. However, most fish will not be affected if the dosages are kept below 0.50 milligram per litre (4.2 pounds per million gallons) and medium-sized black bass can survive in dosages up to 2.00 milligrams per litre (16.6 pounds per million gallons).

There has been some concern expressed by orchard and vineyard owners, whose trees and vine roots have penetrated the drains, that the introduction of copper sulfate will kill the trees and vines. There is little evidence that this can happen with deep drains. Apparently, the absorptive function mechanism of the roots dissipates rapidly and the toxic copper travels only a short distance up the root. This results in only local killing action near the drain and maintenance personnel report there is not even leaf discoloration when the recommended concentrated dosages are used.

Corrugated plastic drain tubing can only be cleaned with high-pressure jetting equipment. As long as the drain has no sharp bends, the jetting equipment follows the pipe. Dead roots up to 12.7 millimeters (1/2 inch) can be cut with a root removing nozzle which has a forward jetting action and rear jets on an angle of 15^{o} to 20^{o} to the longitudinal axis. Some companies now manufacture special closed cage root cutters which have a fixed outer cage and rotating cutters on the inside driven by jet water pressure. For the jetting equipment the pump should be capable of delivering 246 litres (65 gallons) per minute at 6895 kilopascals (1000 pounds per square inch) with a pressure regulator independent of engine revolutions per minute. The nozzle should be capable of supplying 227 litres per minute (60 gallons per minute) at 8275 kilopascals (1200 pounds per square inch) at the pump and 227 litres per minute (60 gallons) per minute) and 4825 kilopascals (700 pounds per square inch) at the end of a 215 metre (700-foot) hose.

The cleaning speed of a jet cleaner is about 305 metres (1000 feet) per hour with few roots and about 245 metres (800 feet) per hour with moderate roots. Cleaning costs vary from 0.33 to 0.66 cents per metre (0.10 to 0.20 cents per foot) for large cleaning jobs to 1.31 to 1.64 dollars per metre (0.40 to 0.50 cents per foot) for short jobs with most of the cost being for mobilization to and from the job. For cleaning out the slots or holes in corrugated plastic drain tubing, holes are drilled in the nozzle at right angle to the longitudinal axis so they cause a jetting action straight into the slot or hole. Jetting equipment can be mounted on trailers, pickups, and large trucks that carry up to 1,000 gallons of water as well as the hose reel and pumps.

All pipe outlets should be inspected in the spring and after heavy rainstorms to determine that the corrugated metal pipe section still has free fall into the open drain and that there has been no erosion on the side slopes which could cause the pipe to be displaced or washed out. When the open drain is cleaned, care should be taken that the cleaning equipment does not crush or displace the outlet pipe. The area of open drain adjacent to the outlet should always be cleaned by hand. If the drain banks are burned, the burner should be raised as it goes over the outlet pipe so the protective coating on the pipe will not be damaged. All pipe outlets should be fenced if cattle and sheep are allowed in the drain area.

Flap gates, when required to keep floodwater in the open drain from backing up into the pipe, should be inspected at least every month. Even the best flap gates tend to stick which could cause the entire system to stop functioning. Where rodent screens have been installed, they should be checked for clogging and displacement at least once during the irrigation season. Where rodent screens have not been installed, the corrugated metal pipe outlet should be inspected periodically during the year for nests. The banks around the outlet should also be inspected for any indications of seepage caused by holes and obstructions in the plastic drain tubing made to protect the nests.

In cold climates, ice buildup at the mouth of the outlet pipe can sometimes cause considerable problems during the winter months. Water leaving the pipe drain is above freezing, but shortly after it leaves the protection of the closed pipe system, and flows over the ice cover in the open drain, it starts freezing and, with flat grades and low flows in the open drain, ice dams are sometimes formed which develop back to the pipe outlet and even into the corrugated metal pipe outlet. Since the bottom and sides of the open drain are usually below the watertable, the bottom and a short distance up the side slope of the open drain do not freeze, and ground water runs below an ice cover even in the coldest climates. Therefore, by providing an insulated and protected passageway from inside the corrugated metal outlet pipe to the bottom of the open drain the subsurface pipe drains can function throughout the year. Since the pipe flows are relatively low during the cold weather season, a 152.4-millimetres (6-inch) diametre flexible plastic pipe wrapped with waterproof insulating material can be pushed back about 1.8 metres (6 feet) into the corrugated metal pipe and the 1.5 metres (5 feet) left extending out of the metal outlet pipe can be bent to direct the drain effluent down toward the bottom of the open drain. This type of outlet should be installed before the water in the open drain starts freezing on the surface. Ice can then build up in the open drain, but no ice dams are built up at the pipe outlet and no matter how cold it gets, the subsurface drain has adequate outlet conditions.

Maintenance at manholes

Manholes are usually constructed at junctions of three or more pipe drains, at locations requiring sharp bends, and at steep grade changes. The bottoms of all manholes are set about 460 millimetres (18 inches) below the invert of the outlet pipe to provide a trap for silt and sand. It is important to keep the silt and sand trap clean and the trap should be inspected and cleaned frequently during the construction and initial operation of the system. Failure to clean the upstream manholes during construction has caused many well-constructed downstream drainlines to become plugged. Special mud and sand pumps have been designed for this work.

Any erosion and settlement around the outside of the manholes should be repaired as soon as it is observed. When nonreinforced concrete pipe is used for the manholes, the joints should be sealed so surface water and soil material cannot channel through the joints. Unless the manhole is expressly designed to receive surface water, farmers should not be allowed to use manholes as surface waste disposal outlets. When flushing a sewer ball through a pipe, heads should not be built up in the manhole more than about 0.6 metre (2 feet) above the top of the inlet pipe. Manhole covers should be fastened down securely at all times, except during cleaning operations and inspections. This is a safety practice to prevent small children and animals from falling into the manhole and others using the manhole to get rid of garbage. Most manholes in agricultural land are safe to enter, but it is a good precaution to use a gas-measuring device before entering a manhole for discharge measurements or setting up cleaning equipment. Some gasses in drains act very quickly and are hard to detect until too late.

Iron bacteria in drains

Iron and mangnese sludges develop in some drains and if not treated the sludge caused by bacteria will gradually fill the water inlet joints, slots or holes in the drain pipe, as well as the gravel envelope material until the drain can no longer function. Affected drains should be jetted about every 2 years depending upon the build up of the sludge in the pipe. A TV-camera inspection will indicate when jetting is required or if a TV-camera is not available, a backhoe can be used to excavate down to the pipe and observations made of the buildup of the sludge in the gravel envelope and in the water inlet areas of the pipe. If the sludge buildup has gone unobserved long enough, usually over 2 years, it sometimes hardens to the degree that jetting alone will not break the sludge out of the slots and holes or affect the sludge in the gravel envelope. A chemical treatment or sulfuric acid must then be introduced into the affected drain to kill the bacteria and soften the sludge so it can be jetted out.

When a chemical treatment is necessary, it should be preceded by jetting to get all loose sludge and other material out of the drain so the chemical can get directly to the plugged water inlet areas. There are a number of methods of introducing the chemical into the pipe and some are extremely dangerous if not handled correctly. One method is to plug the drainline at the lower manhole and fill the affected drainline with a solution containing 0.3 per cent sulfuric acid and 2 per cent sodium bisulfate. This is accomplished simultaneously by introducing concentrated sulfuric acid, water, and a concentrated solution of sodium metabisulfate. This solution should stand in the drain a minimum of 48 hours or longer if the sludge buildup is thick and hard. The manhole is then opened and the solution allowed to flow to the outlet. A second jetting should be started as soon as practical. Using the newly developed "safe" sulfuric acid pellets makes the handling of the sulfuric acid much safer and easier since there is little danger from acid burns on the clothing or the flesh. The sulfuric acid can also be introduced into a sealed section of pipe by the use of sulfur dioxide as a compressed gas at the rate of 0.45 kilogram (1 pound) of sulfur dioxide gas for every 0.0283 cubic metre (1 cubic foot) of tubing volume or at a rate of 0.45 kilogram (1 pound) of sulfur dioxide gas per 28.4 litres (7.5 gallons) of water (Aldrich, 1977). Drains should remain sealed for 48 hours following introduction of the SO_2 treatment.

With the development of "safe" sulfuric acid pellets, it is believed that they can be mixed with the gravel envelope during construction to effectively control the bacteria for the first 2 or 3 years. The use of these pellets will also eliminate the expensive equipment required to introduce the SO_2 gas or liquid sulfuric acid into the pipeline.

Little is known about the environmental aspects of using sulfuric acid in drains. Carefully conducted tests using small fish in control cages anchored in the open drain about 61 metres (200 feet) below the pipe outlet, at the pipe outlet, and 61 metres (200 feet) above the pipe outlet showed as great or greater fish kill in the cage 61 metres (200 feet) upstream from the pipe outlet as the control cage 61 metres (200 feet) downstream. As expected, all fish were killed in the control box at the outlet. Additional testing for fish kill due to introducing sulfuric acid into pipe drains will be conducted as rapidly as possible. However, with due regard for keeping the drainwaters clean and pure for fish and other aquatic life, keeping the subsurface drains functioning as designed by the use of all reasonable maintenance techniques available is essential if modern drainage systems are to be economical, successful, and permanent.

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Paper 4.05

DRAINAGE FOR SALT LEACHING

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Summary

The water flow streamlines can serve as a guide for a first approximation design of irrigation and drainage for salt leaching.The main guidelines are:

a) The different flowtubes should be as much as possible of the same cross-section and best should be parallel.

b) Stagnation areas cannot be leached.

c) To overcome stagnation areas the flow symmetry should be changed from time to time.

d) The most efficient leaching is by vertical infiltration into an unsaturated soil.

e) The quantity of water in one leaching should be equal to or slightly more than the water held in the soil to be leached.

f) A drawdown of water table along with a possible drying up of the soil profile are necessary from time to time to facilitate leaching.

g) The depth of unsaturated soil should be sufficient to accept the amount of water held within the leached profile before the leaching.

The classical formula for leaching requirement is valid only under very limited conditions. More sophisticated studies of solute movement in an active porous medium may be useful for a number of problems and for optimization of the water regimes in irrigated lands. However the major conclusions in the above that are based on immiscible displacement are unchanged.

1. The problem

One of the main purposes of artificial underground drainage is the prevention of salt accumulation or the leaching of already existing salts. Many of the salinity problems arise at areas of higher groundwater table and poor drainage.

The amelioration of saline and alkaline soils or soil conservation are complicated matters. However, invariably they involve leaching out of solutes or the introduction of various chemicals in solution. These are always through the flow of water and is controlled at least in part, by underground drainage.

2. Leaching requirements

The most common formula for leaching requirement maintains that excess irrigation is needed (USDA Handbook 60, 1954).

Consider E - the rate of evaporation, and Q_d - the drainage water. Assume also that C_o is the solute concentration in the incoming water, C_d - the solute concentration in the soil solution. Then, the excess drainage water should be according to the leaching requirement formula

$$Q_d = E \frac{C_o}{C_d - C_o} \qquad (1)$$

There is at least one implicit assumption involved in its derivation that every drop of water and salt are completely and thoroughly mixed in the whole soil volume before they are drained. Any similarity between this formula and the actual happening in the field is accidental. To prove this extreme statement it is sufficient to show one or two very common ways of salt leaching where the salt distribution in the soil is not sensitive to the quantities of water used. Alternatively it is sufficient to show that different methods of leaching will lead to widely varying results using the same quantity of leaching water or require extremely different water quantities to obtain the same result.

The lengthy disapproval of this famous leaching requirement formula would not be necessary if not for the wide and prolonged adherence to it which is supported by intuition. This intuition relies on the general feeling that more leaching water is necessary if there is more evaporation, if the initial salt concentration is high, and the salt tolerance is low. This trend is probably true for a given leaching configuration but it is secondary to the determination of the configuration itself.

In the following, we shall suggest a simple approach to study the leaching configuration and a key for a design.

3. The leaching process

The movement of liquids in porous media have been studied extensively by many: Bear et al. (1968), Bear (1972). In principle it involves two main flow components.

a) Mass flow of the flowing water with the solutes in it, sometimes referred to as "piston" flow, sharp leaching front or immiscible displacement.

b) Hydrodynamic dispersion or eddy diffusion relative to the average position of the leaching front.

The two processes are simply superimposed at any given moment.

One should not minimize the importance of the second process of hydrodynamic dispersion in the leaching process. Exchange mechanisms between the soil solution and the soil are also quite important (e.g. in strongly aggregated soil at unsaturated flow). Nevertheless, it will be very convenient to consider first the motion as that of water (or immiscible) displacement only and only later superimpose other motions. As will be shown in the following, this approach will lead to some very simple rules for drainage design. Such simplicity is essential for the engineering way of thinking.

4. The leaching front in a flow net

Given a flow net, one can draw the movement of the average leaching front in a very simple way (see appendix for the definition of this front). In our first approximation it is imagined that before the front there exist the high salt concentration and behind it the soil is leached and has a low salt concentration. Assume the moisture content to be θ (on a volume basis) and the water flux to be $\underline{q}$. The velocity of the front $\underline{V}$ would then be

$$\underline{V} = \underline{q}/\theta \tag{2}$$

A flow net is usually drawn in such a way that the discharge Q_i in every flow tube as defined by neighbouring streamlines is the same. The leaching front (e.g. in a two dimensional cross section) within a simple flow tube sweeps an area A_i per unit time

$$A_i = Q_i/\theta_i \tag{3}$$

As Q_i is the same in every flow tube, equal areas are being leached per unit time in each flow tube. This is a very simple rule of thumb with respect to leaching.

Figures 1, 2, 3 and 4 show the advance of the leaching front due to drains at a depth y and half spacing L in a deep uniform soil with water ponded at the surface. It has been solved by an electrical analog. Figure 5 indicates the leaching towards a ditch half filled with water (using a Hele-Shaw model as a tool for solution).

Clearly the narrowest flow tube is leached fastest. As soon as the fastest streamline has been swept through by the leaching front the flow tubes of the smallest cross-sectional area have been leached. The water that continues flowing through these small flow tubes is wasted and does not contribute towards further leaching. One can define the marginal efficiency of added leaching f_t as follows:

$$f_t = \frac{\text{added volume of leached soil per unit time} = \sum_k^r Q_i/\theta_i}{\text{potential volume that could be leached} = \sum_o^n Q_i/\theta_i} \tag{4}$$

where k flow tubes have already been traversed by the leaching front.

Figure 6 showed the fast reduction in marginal leaching efficiency.

Figures 7, 8 and 9 show the leaching front as found by a Hele-Shaw model in ponding the whole surface, 2/3 of the surface and 1/3 of the surface respectively. Fig.10 shows the leaching efficiency in these three cases.

So far we can draw a number of important conclusions. First it is clear that with the same water quantity different extent of leaching will be obtained for different drain spacings. Furthermore, it becomes obvious that no amount of water will leach a large stagnation area between the drains. At least as a first approximation the leaching requirement formula has nothing to do with the actual physics of flow. To obtain leaching, the leaching front has to traverse the flow tubes. It is irrespective of the salt concentrations involved.

A partial ponding of the field has improved the leaching efficiency (Figures 7, 8 and 9).

Another way to improve the efficiency of leaching was found by irrigating or ponding the field in several stages. The ponding starts over the whole field for a short time and then recessed step by step towards the field centre. Figures 11 and 12 show 3 and 5 steps leaching as found by a Hele-Shaw model. Figure 13 compares the efficiencies of 1, 3 and 5 stages leaching.

5. Leaching configuration - rules of the thumb

1) Flow tubes of highly different cross-sectional areas lead to low leaching efficiency.

2) The highest efficiency is obtained by parallel streamlines. (e.g. vertical flow).

3) Stagnation zones and stagnation areas cannot be leached.

4) To leach stagnation areas one has to change from time to time the symmetry of the flow in one of the following ways:

- wetting part of the field;
- introducing water in one drain and letting it out in another;
- changing the place of drains (e.g. shallow drainage furrows that are seasonally renewed);
- closing off alternative drains.

It is clear from figures 1-6 in water logged areas and without any leaching "tricks" the drains can leach efficiently only as far horizontally as their depth or roughly to that distance. It would, however, be a poor reason to increase the depth of drains or reduce their spacing.

6. The most important leaching trick, infrequent soil drying

At first sight the above summation does not make one very optimistic about the usefulness of drains in leaching. However, field observations somehow indicate to the contrary. Two main conclusions may be reiterated here.

a) The most efficient leaching from a hydraulic point of view would be with parallel streamlines or flow tubes of even areas. This could be obtained more or less by irrigation into an unsaturated soil and smooth soil surface and uniform slope.

b) Leaching is obtained, as a first approximation if the leaching front sweeps through a part of the flow tube which is to be leached.

The first conclusion suggests the basic configuration. The second conclusion gives one a first approximation basis for estimating the leaching requirement.

Consider a soil column with a vertical distribution of moisture as schematically described in Figure 14. The solute distribution before irrigation may be any at all. The top soil is swept through by the total irrigated water. Lower horizons are also swept through by water that is sufficient to exchange the local moisture more than once. Thus, the solutes are materially leached to the bottom part of the soil at the end of the zone of moisture fluctuations. This happens every time the soil is rewetted.

One can now formulate the leaching requirement as follows: The soil profile should be drained and dried frequently enough and to a sufficient depth so that the solutes will be leached beyond the zone where they can cause harm.

7. The leaching requirements in a vertical flow

Consider a rootzone of some depth with the moisture before leaching being θ_d and its total

$$W_d = \int \theta_d d \tag{5}$$

Let us assume that W_d is equal to the field capacity W_f or smaller ($W_d \leq W_f$).

Leaching by an immiscible displacement requires that the smallest moisture quantity to pass and come out of the rootzone is W_d. However, there can be no water flow of moisture lower than the field capacity W_f. Therefore, at every leaching to the bottom of the rootzone more water has to be added to make up to the water depth of field capacity W_f. Thus the leaching water at one leaching operation has nothing to do with the rate of evaporation or the salt tolerance, salt distribution or the water quality. Rather it is equal to the moisture of field capacity within the rootzone. This is true at least as a first approximation.

Now let us consider a number of complete leachings in one season. If the rate of evaporation is E over a period T then the average salt concentration within the rootzone is

$$C_d = C_o + \frac{E\ T\ C_o}{W_d} \qquad (6)$$

where C_o is the initial concentration in the leaching water and W_d the lowest moisture before irrigation. By simple arithmetics.

$$T = \frac{C_d - C_o}{C_o} \frac{W_d}{E} \qquad (7)$$

and the average daily excess leaching water becomes

$$\frac{W_f}{T} = \frac{C_o}{C_d - C_o} E \frac{W_f}{W_d} \qquad (8)$$

This formula reminds us very much of the leaching requirement formula (Eq.1). Thus, if there is thorough leaching several times a season, the leaching requirement by Eq. (1) is only roughly correct. It is increased by the ratio W_f/W_d, the field capacity to the dry state before irrigation.

A regular irrigation quantity W_i will displace a soil volume which roughly contains this volume before the irrigation. Thus a more thorough leaching is obtained by larger irrigation rations and drier soil before irrigation. A little excess moisture each irrigation can be effective or ineffective depending on the salt movements up and down between irrigations and the quality of water that leak below the rootzone.

To leach the rootzone the leaching water W_f should be stored below. Therefore, drainage or drying before leaching should be to a depth larger than the rootzone. The actual extra depth of a watertable should be determined by the actual moisture distribution above it and the deficit to saturation.

In the above, we did not exhaust the problem of vertical leaching into unsaturated soil. First hydrodynamic dispersion will cause a gradual change in salt concentration rather than an abrupt one at the leaching front.

The concentration of the leaching front is somewhere between the average soil solution concentration and the leaching water concentration. To get a lower concentration of the leaching front, more water has to be passed than the mere water depth of field capacity.

The exact leaching regime with dispersion and salt or ion adsorption and exchange, may be computed. Such detailed computations do not change the basic directives obtained by our first approximation, by the assumption of immiscible displacement.

Some quantitative changes are possible but not the choice of approach or method. The exact quantitative determination should be in the eventual field practice anyhow.

The use of small excess quantity of water in frequent irrigations is very complicated. It should be studied by simulation using an electronic computer.

The leaching efficiency dependence on the degree of saturation soil structure and the rate of leaching are also an important subject for physical studies.

We may nevertheless summarize by stating that the most effficient leaching is by uniform flow tubes. This is achieved by irrigation into unsaturated soil. Drainage is then necessary to allow for a sufficient drawdown before leaching. The minimum depth is that of the rootzone plus a certain depth to receive the displaced salty water. The quantity of leaching water is about the volume of water at field capacity to the depth of rootzone.

8. Other considerations about the leaching depth

Large enough depth of leaching is necessary for two purposes:

1. To prevent, in some plants, the contact of even a part of the roots with a saline zone;
2. To prevent the rise of saline solution into the rootzone during period of soil drying.

It should be made clear here that there are several types of fronts in the soil that can move independently:

a) A certain concentration level. It can move up and down due to an actual water flow or by evaporation or adsorption of water into the roots, and by leaching.

b) A certain surface of water particles. Its movement is only by flowing water. It is this movement upward that we wish to avoid so that salts that have been leached will not move upward.

c) The phreatic surface or zero pressure, also called the watertable. This surface can change its place independently. Its upward movement can take place by two processes:

- upward water flow;
- accreation of water from top.

The first may be harmful from a salination point of view, the second not. A drawdown of the watertable can also occur in two ways:

- a downward flow that exceeds the waterflow from the top;
- an upward flow and evapotranspiration is not affected by an artesian flow.

Clearly the first process may be beneficial for leaching while the last may be harmful.

The position and change in the watertable cannot serve as a unique criterion for salinity conditions. Various terms like critical depths for salinity of watertable are therefore a priori ambiguous. Rather, one should try in each case to maintain a slight downward component of flow on one hand, and avoid situations of upward flow of saline water. A fourth type of front in the soil is that of zero vertical gradient, above it the flow is upward and below it the flow is downward. We wish that the front of such zero flow will be above the zone of the stored saline water. Often this may be achieved by loading fresh water and lowering the watertable between periods of leaching. One should note the benefit in fluctuating watertable. At steady watertable the rate of vertical flow at midpoint between drains may be quite negligible. Therefore the rate of downward leaching may be extremely slow (as in the Figures 1-6). However a drawdown may be obtained even at the mid point by lateral flow. Through such flow the saline water displaced by vertical leaching in the unsaturated zone will be gradually washed out.

In summary, the drawdown of the watertable is necessary to facilitate the leaching in the unsaturated soil. The depth should be at least that of the rootzone plus the space to store temporarily the displaced water. A large drainage depth may be required to maintain a low upward water flow under certain circumstances. No general simple rules can be drawn here for such depth.

A fluctuation of the watertable by repeated irrigations and drainage periods is more efficient in salt leaching than continuous steady flow under a fixed watertable.

9. An example of heavy soils in semi-arid zones

Consider, for example, heavy cracking soils (highly montmorillonitic in a semi-arid zone). Shallow and dense drainage is needed for aeration purposes and improved trafficability as only the top 25-40 cm are permeable. This top layer is also responsible for most of the vegetative activity. Deeper drains may run into problems because the soil surrounding them is highly impermeable, or tends to lose its permeability within few seasons by repeated drying and wetting. The shallow drainage must be at a spacing of 20-30 m (or less). Most of it can be obtained by deeper furrows or between elevated beds. Deeper drains (1.5-2 m) may be installed at a spacing of 200-300 m as main collectors and to provide for one or two good drawdowns in a year for leaching purposes. It was proven sufficient if a drying of the soil at the late summer takes place. In the absence of high regional watertable there may be a slight deep leakage of less than one millimeter per day. As small as it is, it can act well to remove the salts to greater depths without any deep drains.

In several projects in semi arid zones, alluvial grummosols (heavy montmorillonitic clay soils) have been observed especially where high watertables exists. Natural drying in summer can reach about 1.5 metre depth. The rain that follows, displaces the salts into that depth. Often its salt concentration can reach sea water quality with no harm to crops. In places of higher watertables and in some cases even under artesian conditions, it has been observed that a gully of 2-3 metres depth will often maintain a lower salinity to a distance of five hundred metres on each side.

10. Uniformity of the moisture region and salinity

It is stated elsewhere (Zaslavsky, 1978; Zaslavsky & Sinai, 1977; Zaslavsky 1970) that horizontal flow of water near the soil surface occurs even when the soil is not saturated. Moisture will tend to accumulate in concave spots due to raindrop splashing due to flow in the top more permeable soil (the transition layer to the air) and in layered soil. These phenomena along with regular runoff will cause extreme non uniformities of the moisture regime. If the accumulation of moisture in concave places is moderate it may be sufficient to cause excess salinity but not enough to leach the soil. In other cases leaching water may be distributed unevenly over the field. In measurements and calculations it has been shown that local water recharge can be several times the average while in other parts it may be only a fraction of the average. Accordingly, the leaching of salts will not be uniform. The details of the flow regime are beyond the scope of this article. However it may be briefly noted.

- If the net recharge is not uniform the leaching is wasteful in time and in water quantities.
- A plane field with no concavities with a uniform cultivation will have a uniform leaching
- In an unsmoothed field, there is some compensation in the sense that more salt is accumulated in the vicinity of a concave region and more water tends to pass and accumulate there during the leaching operation.

11. Concluding remarks about the leaching geometry

In the above an attempt was made to provide the basic rules for judging a drainage design for salt leaching. Intentionally, we refrained from complicating them with the superposition of dispersion (longitudinal and lateral). The problems of exchange in saturated and unsaturated flow and the isotropy of flow were also overlooked. These and other complicated subjects are at least in part available in the various scientific writings but they cannot change the simple notions suggested in the above as the dominant design rules.

These design rules are:

a) Keep the flow tubes as uniform and as parallel as possible.

b) Break the flow symmetry to avoid areas of stagnation or change the stagnation place from time to time.

c) Utilize changes of soil moisture for leaching.

d) Leaching is obtained by flowing water through a part of a flow tube just a little more than the moisture contained within that part of the flow tube.

e) One of the most efficient ways of leaching is by vertical flow into a non saturated soil. The depth of unsaturated soil must be greater than the rootzone to accept the displaced water.

f) The water volume in one vertical leaching is roughly the moisture at field capacity within the rootzone.

g) In case that several leachings are necessary within a season the frequency of leaching will depend on the rate of evaporation and on the salinity tolerance. The total water requirement will then be the product of this frequency and the volume per leaching.

h) Periodic changes in watertable are better for leaching than a steady state watertable.

i) For most efficient leaching the field should be plane, smooth and uniform.

The above can serve every inventive drainage engineer to apply his own "tricks" for his own specific needs.

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Figures

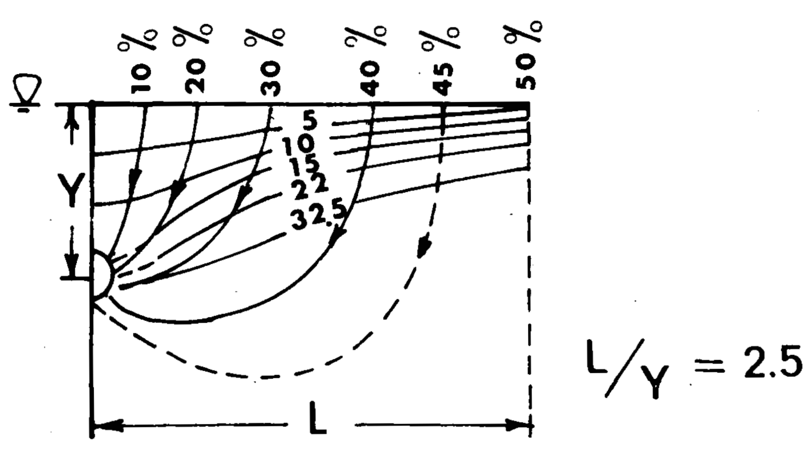

Fig.1. Leaching by ponding. L/Y = 2.5. Found by an electrical analog and planimetry.

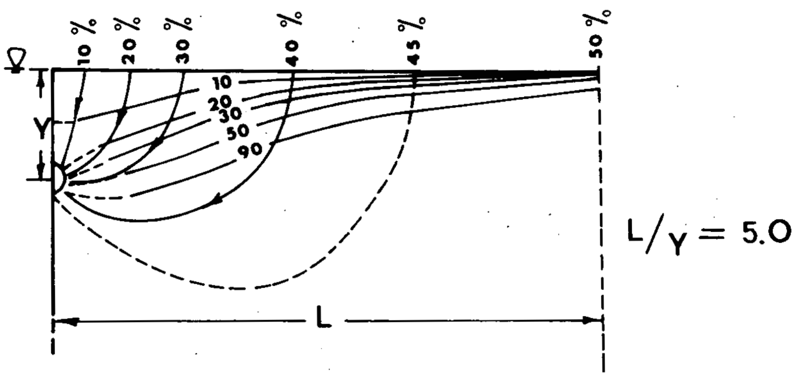

Fig.2. Leaching by ponding. L/Y = 5.0. Method as in Fig.1.

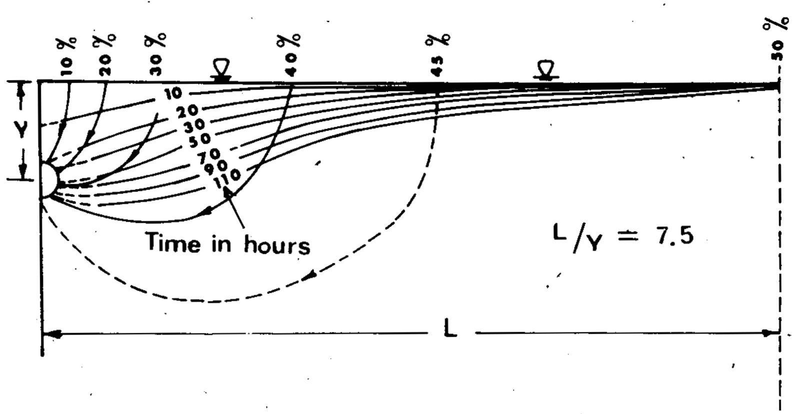

Fig.3. Leaching by ponding. L/Y = 7.5. Method as in Fig.1.

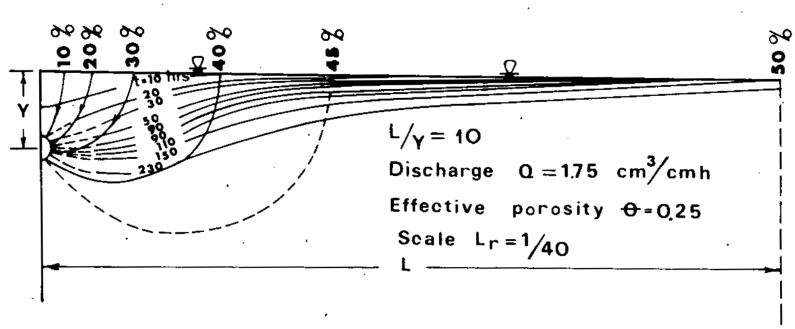

Fig.4. Leaching by ponding. L/Y = 10. Method as in Fig.1.

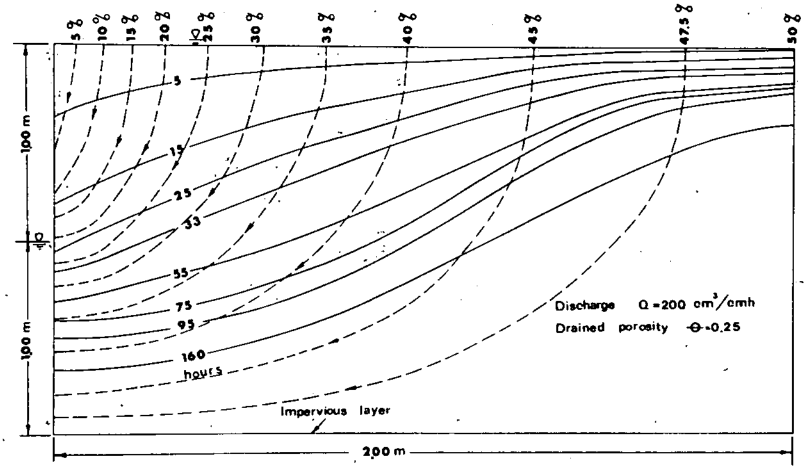

Fig.5. Leaching by surface ponding into a half filled drainage channel. Streamlines (noted by per cents) were obtained by Hele-Shaw model. Front lines (noted by hours) were obtained by planimetry, and validated in the model.

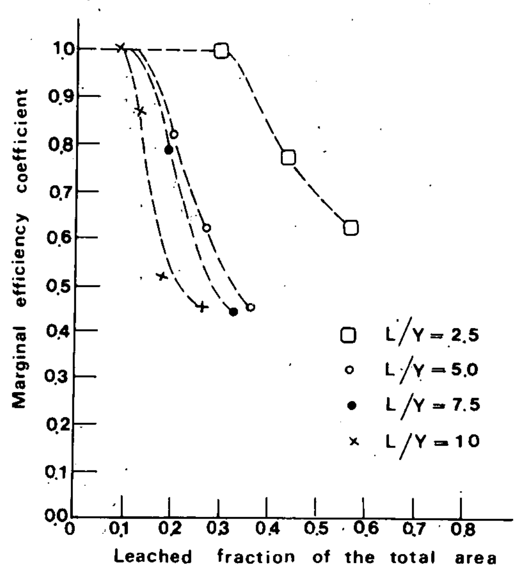

Fig.6. The relation of the marginal leaching efficiency (f_t) to the leached fraction of the soil drains of depth Y and spacing 2L with ponded water (related to Figs. 1-4).

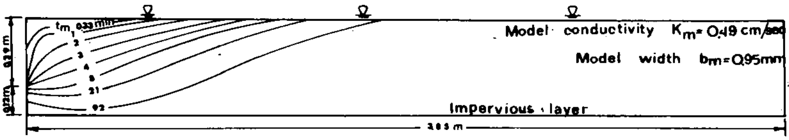

Fig.7. *Leaching to drains by ponding whole the surface. Front lines from the Hele-Shaw Model.*

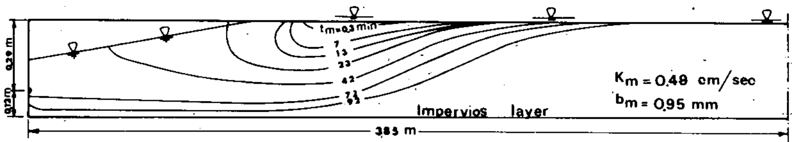

Fig.8. *Leaching to drains by ponding 2/3 of the surface. (From Hele-Shaw Model).*

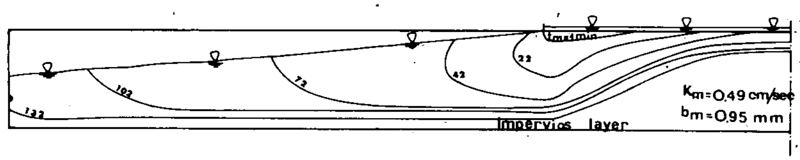

Fig.9. *Leaching to drains by ponding 1/3 of the surface. (From Hele-Shaw Model).*

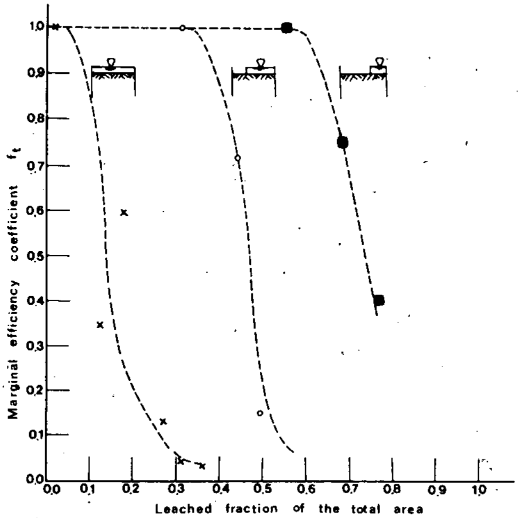

Fig.10. *Efficiency coefficients calculated from Figures 7, 8 and 9.*

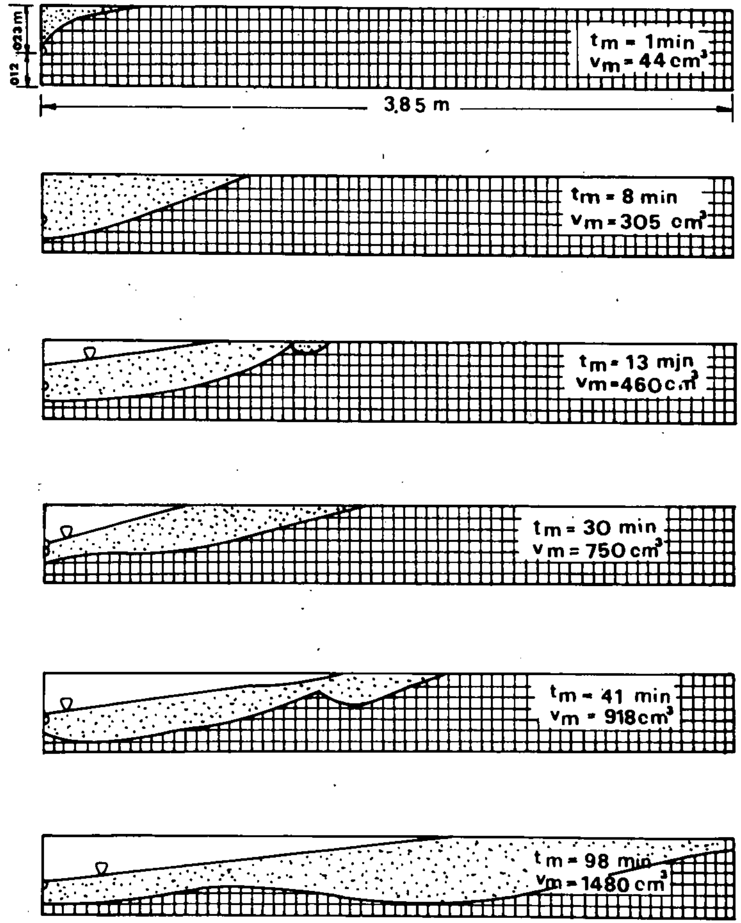

Fig.11. Leaching by ponding in 3 stages: tm=time in the model, Vm=volume of leaching liquid:Dotted area fresh water.

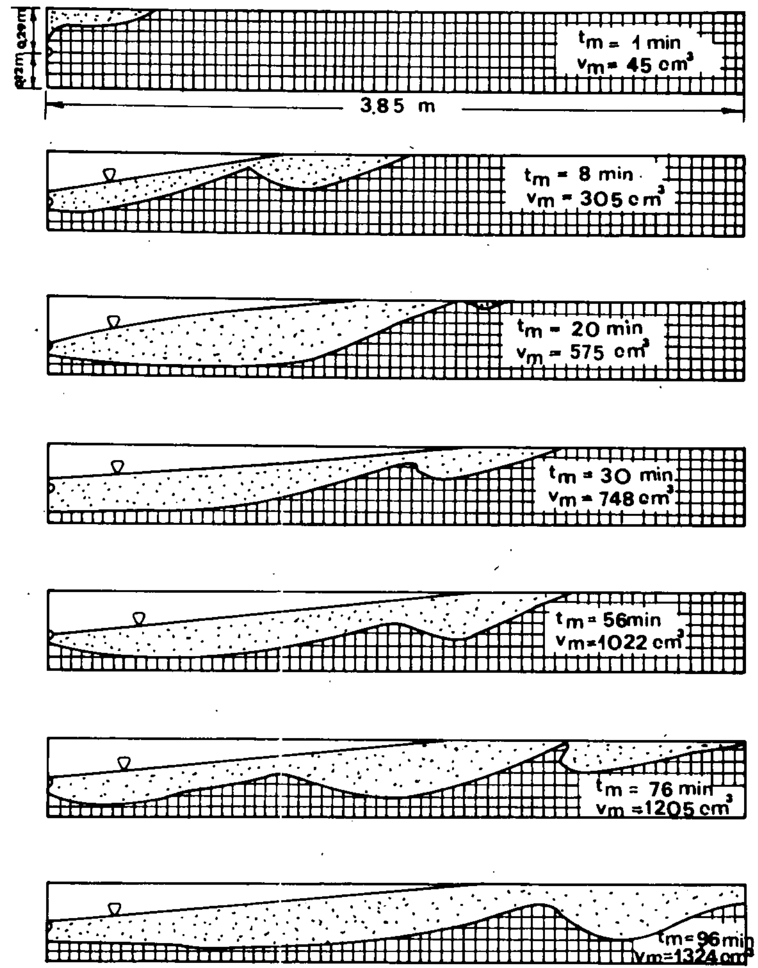

Fig.12. Leaching by ponding in 5 stages: tm=time in the model, Vm=volume of leaching liquid:Dotted area fresh water.

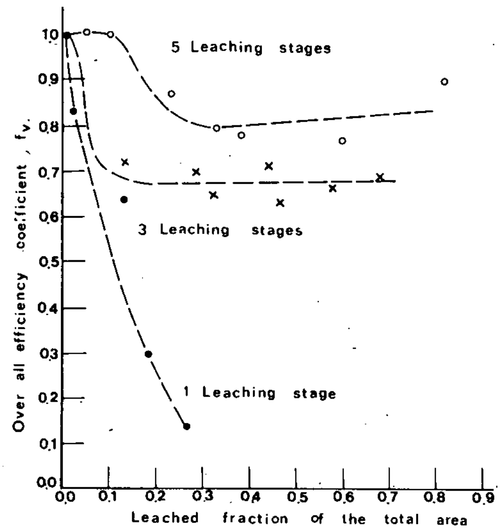

Fig.13. Efficiency coefficients for various methods of leaching.

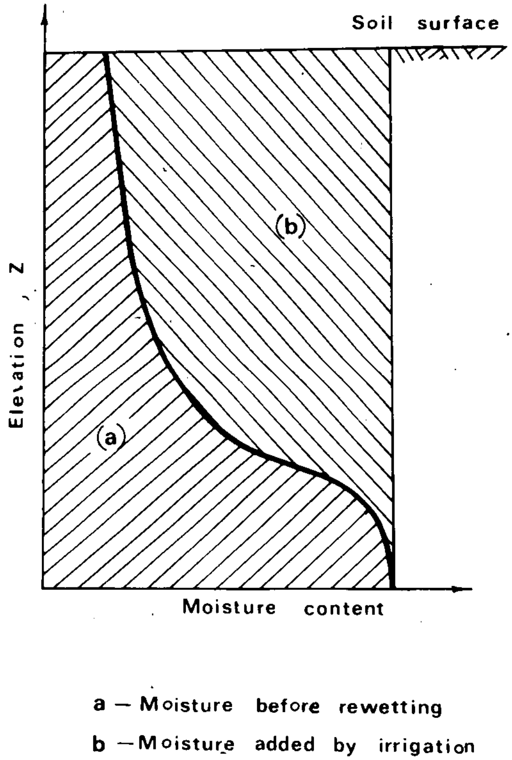

Fig.14. Moisture distribution in a soil profile-schematic.

Appendix

Leaching front

Consider the following equation of flow

$$q_t = C\theta V_s + Cq_R + q_d \tag{1}$$

where q_t is the total flux vector of a solute species; C the concentration of the given solute in the soil solution; θ the volume content of solution in soil; V_s the movement velocity of the solid matrix; q_R the flux of the soil solution relative to the solid matrix; q_d - the flux of the solute relative to the soil solution due to dispersion and diffusion.

The equation of dispersion is stipulated as (also proved experimentally)

$$q_d = - D_h \text{ grad } C \tag{2}$$

Neglecting soil deformation ($V_s = 0$)

$$q_t = Cq_R - D_h \text{ grad } C \tag{3}$$

conservation requires

$$\text{div } q_t = \frac{\partial}{\partial t} (\theta C) - \frac{\partial S}{\partial t} \tag{4}$$

where (θC) is the solute content in solution and S is the content of the same species in an adsorbed position or otherwise inaccessible for direct mass flow or as measured in the soil solution.

Combination of (3) and (4) gives the well known equation

$$- \text{div } Cq_R + \text{div } D_h \text{ grad } C = \frac{\partial}{\partial t} (\theta C) + \frac{\partial S}{\partial t} \tag{5}$$

The equation is better known and simpler to use for cases where $\frac{\partial S}{\partial t} = 0$. Consider a coordinate x in the direction of waterflux q and define a new coordinate $\zeta = x - \frac{q}{\theta} t$

Eq.(5) reduces to a simpler, one dimensional equation

$$\frac{\partial}{\partial \xi} (D_h \frac{\partial C}{\partial \xi}) = \theta \frac{\partial C}{\partial t} \tag{6}$$

which is a diffusion type equation around a reference ($\xi = 0$) which is moving at the rate of (q_R/θ). This is actually the velocity of the central leaching front in a one dimensional flow in a uniform medium with no adsorption and exchange, no time and space dependent sinks and sources. The development in the main text was related to $D_h = 0$ at Eq.(6) which means an immiscible displacement.

A more general way of following a front of a certain property is by taking the total derivation of a property P(xyzt) = (a)

$$\frac{dP}{dt} = \text{grad } P.V + \frac{\partial P}{\partial t} = 0 \tag{7}$$

where (a) is a constant value of the property P and V is the velocity of the front normal to the front

$$(1_x \frac{dx}{dt}, 1_y \frac{dy}{dt}, 1_z \frac{dz}{dt})$$

For the special case of steady flow with no sinks and sources in a uniform soil it may be shown that if P in Eq.7 is taken as a sharp concentration change that moves at the velocity (q_R/θ), as inflection point

$$(\frac{\partial^2 C}{\partial x^2} = 0)$$

or in other cases the maximum concentration

$$(\frac{\partial C}{\partial x} = 0)$$

the velocity V is also (q_R/θ). In the simpler cases the concentration at the inflection point is the median between the initial and final concentration (before and after the leaching).

Assuming that the water flow is independent on the salt concentration pattern, it can be solved by well known methods. From the flow regime the concentration pattern may be computed as a diffusion equation with a convective term from (5)

$$- C \text{ div. } q - \text{grad } C.q + D\nabla^2 C + \text{grad } D.\text{grad } C = \theta \frac{\partial C}{\partial t} + C \frac{\partial \theta}{\partial t} + \frac{\partial S}{\partial t} \tag{8}$$

q(xyzt) and θ(xyzt) in (8) the water flow equation itself is

$$-\operatorname{div} q = \frac{\partial\theta}{\partial t} \tag{9}$$

and (8) can be reduced by subtracting from both sides of (8) $C \operatorname{div} q = C(\partial\theta/\partial t)$

$$-\operatorname{grad} c.q + D\nabla^2 C + \operatorname{grad} D.\operatorname{grad} C = \theta \frac{\partial C}{\partial t} + \frac{\partial S}{\partial t} \tag{10}$$

This equation describes the motion of the solute as a superposition on a known water flow.

The hydrodynamic dispersion coefficient D depends mainly on the hydraulics of flow. (We assumed in the above to be a scalar but in fact it is strongly non isotropic). Nevertheless in many problems the product grad D.grad C is extremely small compared for example with C.q or even with $D\nabla^2 C$. This is due not only to the fact that it is a product of two small terms but due to the possibility that grad D is nearly or exactly normal to grad C.

Grad D is anyhow supposed to be a known factor. The point which is being discussed above is that a second approximation solution can be obtained by the equation

$$-\operatorname{grad} C.q + D\nabla^2 C = (\theta + \alpha) \frac{\partial C}{\partial t} \tag{11}$$

with a linear absorption isotherm $S = \alpha C + \beta$. The solution is then seen as a superposition of the mass motion $q/(\theta + \alpha)$ and the dispersion

$$D \frac{\partial^2 C}{\partial \xi^2} = (\theta + \alpha) \frac{\partial C}{\partial t} \tag{12}$$

assuming ξ to be a curvilinear coordinate parallel everywhere to the water flux q.

The above can be proved more systematically by assuming a solution

$$C = C_o + EC_1 + E^2 C_2 + \ldots$$

where E is an arbitrarily small coefficient and similar terms for the flux q and the moisture content θ. The solution is found for successive terms in powers of E.

The essence of these corrections is that the concentration near the leaching front is around the median between the initial and the final concentration. Letting the leaching progress further, there will be formed a lower concentration. Requiring lower concentration (closer to the leaching water concentration) calls for further movement of the leaching front.

A more detailed treatment of the diffusion equation and leaching is beyond the scope of this article. Furthermore, it is of a secondary importance to the engineer when deciding about fundamentals of the technical field solution.

Paper 4.06

THE EFFECT OF CAPILLARY FLOW ON SALINIZATION AND THE CONCEPT OF CRITICAL DEPTH FOR DETERMINING DRAIN DEPTH

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Summary

Only where seepage occurs can capillary flow contribute to salt input in the rootzone. Where no seepage occurs, capillary flow soon ceases because of the fall of the watertable and a mulching effect of the surface layer. The critical watertable depth is defined as the depth to which the watertable will fall in the absence of seepage and at which capillary rise is reduced to almost zero.

If seepage flow cannot be eliminated or intercepted, the drain depth should be calculated as the sum of the critical depth and the hydraulic head necessary to discharge the seepage flow to the drains. In the absence of seepage or in the case of seepage of fresh water, the critical watertable depth is not a determining factor for drain depth.

1. Introduction

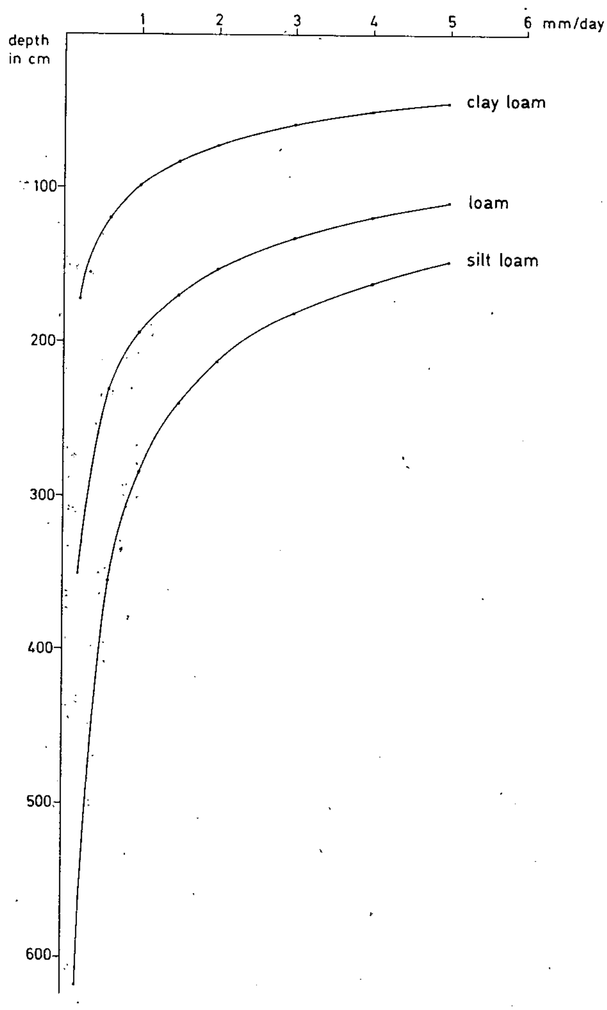
Arguments often used in favour of deep drainage in irrigated areas are the following:

- a great drain depth allows a wide drain spacing, as long as the higher cost of digging deeper drains is offset by the lower cost of wider spacings,
- it provides a greater safety margin to cover the water losses that are caused by a low irrigation efficiency and are difficult to estimate,

- deep drainage reduces salinization by capillary flow.

Considerable misunderstanding exists on the subject of salinization due to capillary flow. The argument for applying deep drainage to reduce it is often used in cases for which it is not valid, as will be explained below.

Fig. 1 Relation between capillary flow velocity and depth of watertable for a suction of 16 bar at the surface.



2. Capillary flow from a constant watertable

Figure 1 presents the relation between capillary flow velocity and depth of the watertable for three soil types, ranging from clay loam to silt loam, with a suction of 16 bar assumed at the surface. The data (Annex 1) were calculated and published by Rijtema (1969).

If the watertable remains at a constant level as a result of seepage inflow or, as a special case in The Netherlands, by subirrigation, the capillary rise is considerable, e.g. 180 mm for a period of 6 months, that is 1 mm/day if the watertable remains at a depth of 1 m for a clay loam, at 1.95 m for a loam, and at 2.85 m for a silt loam. These values show the clear difference between soil types; higher silt content causes more capillary rise.

If the seepage water is saline, this capillary rise means a severe increase in the salt content of the rootzone, especially in the surface layer, where the water evaporates.

The depth of the watertable at which the capillary flow velocity reduces to 1 mm/day is often defined as the critical depth. Where seepage occurs, this definition of critical depth cannot be used for determining the depth of drainage because, by admitting a capillary flow velocity of 1 mm/day, the salinization hazard would still be too great. Since the salinity of groundwater is usually much higher than that of irrigation water, a salt input from a capillary flow of 1 mm/day would even exceed the salt input from irrigation water.

In reality, however, the assumption of a gradual increase of the suction towards a value of 16 bar at the surface will not hold true. Since, under semiarid or arid conditions, the evaporation generally exceeds the capillary flow velocity, a surface mulch develops. The moisture transport in the vapour phase through the mulch layer will then become the limiting factor for capillary flow, reducing it considerably.

Nevertheless, if the upward capillary flow is not counterbalanced by a downward flow of leaching water, capillary rise fed by seepage may salinize the soil profile in the long run, even in the case of low watertables. In this respect the Punjab offers unmistakable examples of salinization due to capillary rise in non-irrigated areas fed by seepage water from neighbouring irrigated areas. The irrigated areas show higher watertables, e.g. at a depth of 1.50 m, and a soil profile in which salinity increases with depth, whereas the non-irrigated, salinized areas show a lower watertable, e.g. at a depth of 4 m, and a soil profile in which salinity increases towards the surface.

If the soil profile is composed of several layers, the depth from which capillary flow reaches the surface layer may be greater than in a homogeneous soil profile as has been shown by Varallyay (1974).

The following conclusions can be drawn:

Because of the presence of a surface mulch, capillary rise from a constant watertable will be less than that calculated according to a steady-state concept and the assumption of a gradual increase of the suction towards the surface. In the long run, however, it may still lead to a secondary salinization it if is not counteracted by a downward movement of leaching water.

Where seepage occurs, the often employed definition of critical depth as the depth at which the capillary flow is 1 mm/day cannot be used for determining the depth of drainage.

3. Capillary flow from a falling watertable

If capillary flow is not fed by seepage, the flow will cause the watertable to fall. This fall and the capillary rise during the fall can be calculated from the data presented in Annex 1, assuming again a gradual increase of the suction to a value of 16 bar at the surface.

Figures 2a, 2b, and 2c show the relation between depth of watertable, moisture content, and capillary flow velocity for a suction of 16 bar at the surface. This can be illustrated in a loam profile in which a fall of the watertable from 153 cm to 169 cm means a change in the moisture profile of 28 mm. As the average capillary flow velocity is approximately 1.75 mm/day, it will take 16 days for the moisture being displaced by capillary flow to reach the surface and for the watertable to drop 153 cm to 169 cm.

Fig. 2 a Relation between depth of watertable, moisture content and capillary flow velocity for a suction of 16 bar at the surface.

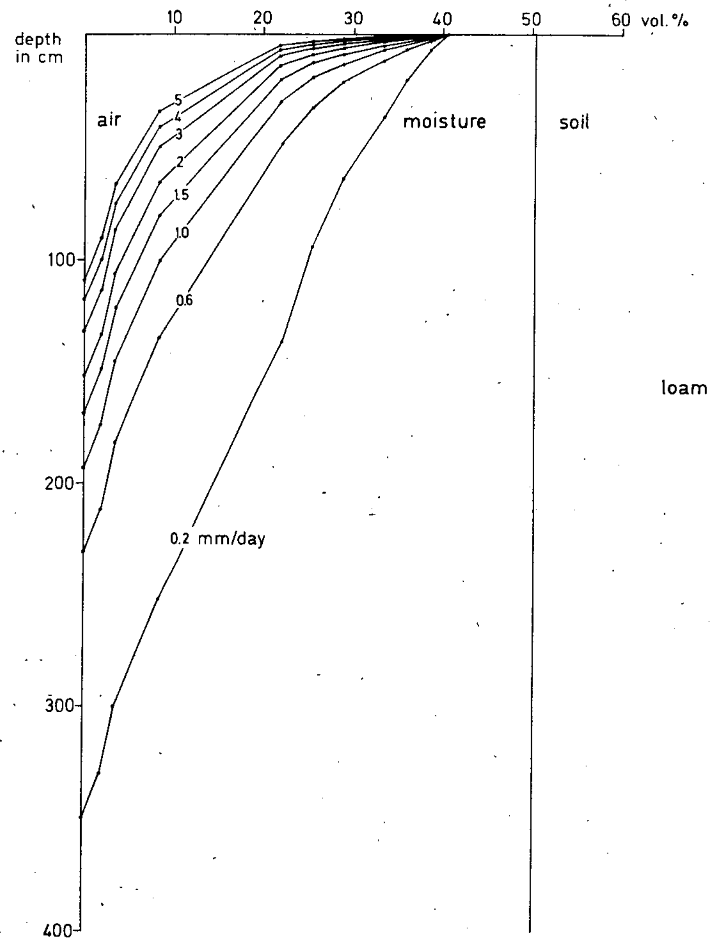


Fig. 2b Relation between depth of watertable, moisture content and capillary flow velocity for a suction of 16 bar at the surface.

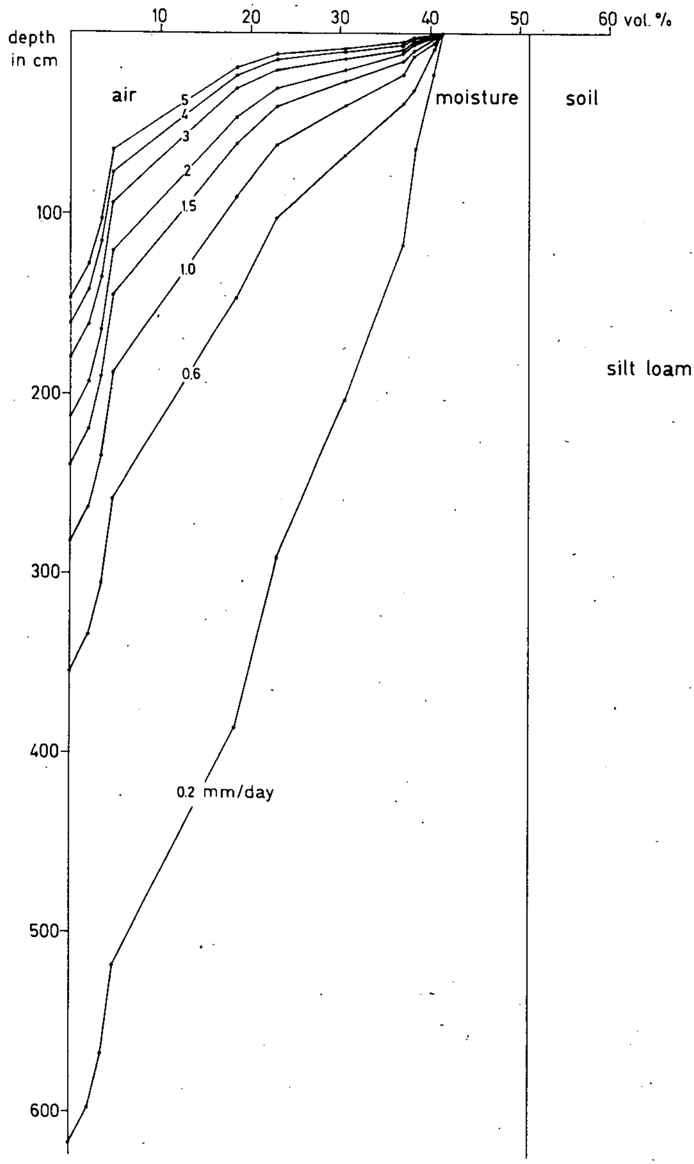
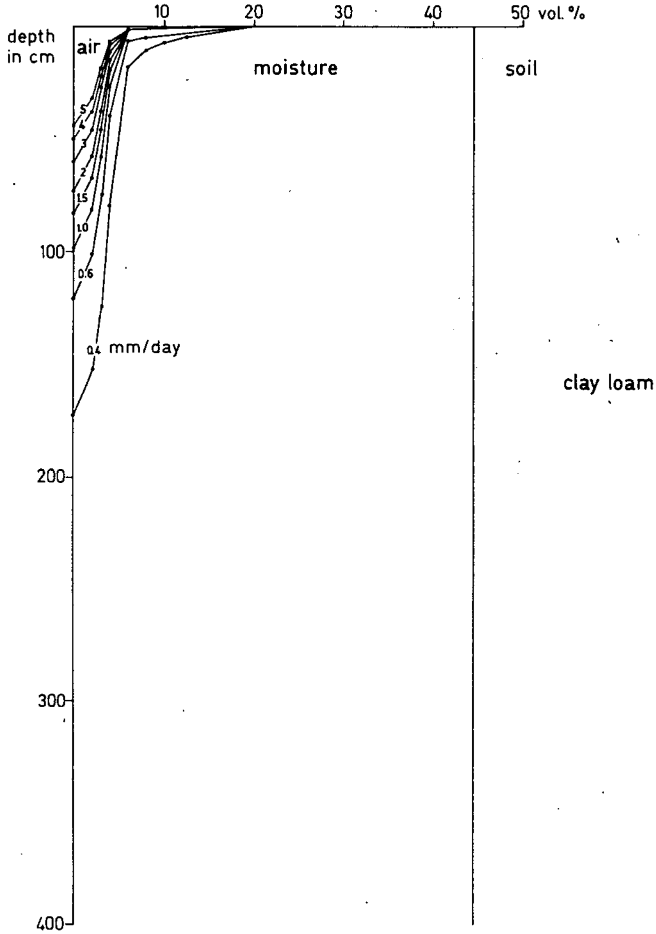


Fig. 2c Relation between depth of watertable, moisture content and capillary flow velocity for a suction of 16 bar at the surface.



In this way the fall of the watertable and the amount of moisture displaced towards the surface can be calculated for a fallow period of six months.

Figure 3 shows than only a small part of the moisture is provided by the soil profile below the initial watertable. The result is summarized in Table 1 and in Figures 4 and 5.

TABLE 1. Fall of the watertable and amount of moisture displaced by capillary rise during a six-month fallow period

Initial water table below surface (cm)	Soil type	Fall of water table (cm)	Average cap. velocity (mm/day)	Total cap. rise (mm)	Ratio cap. rise/fall	Part.cap.rise (from profile below water table)	Ratio part.cap. rise/fall
100	clay loam	90	0.3	50	0.055	25	0.028
	loam	130	1.3	230	0.175	75	0.058
	silt loam	190	2.0	350	0.185	140	0.074
150	clay loam	50	0.2	35	0.07	10	0.02
	loam	85	1.0	170	0.20	30	0.035
	silt loam	145	1.7	300	0.205	70	0.048
200	clay loam	25	0.1	20	0.08	5	0.02
	loam	55	0.7	120	0.22	10	0.018
	silt loam	105	1.3	230	0.22	30	0.029

Fig. 3 Moisture displaced by capillary rise in the case of a falling watertable.

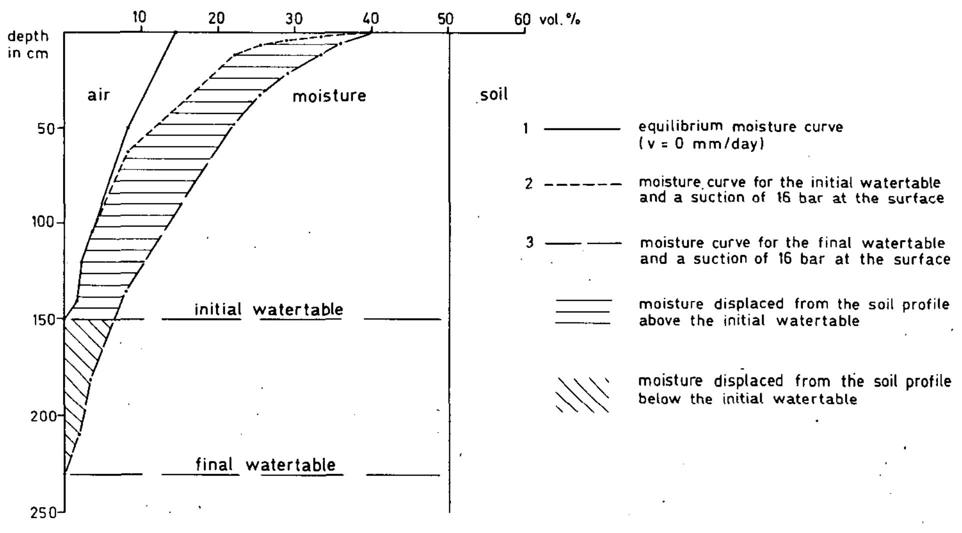
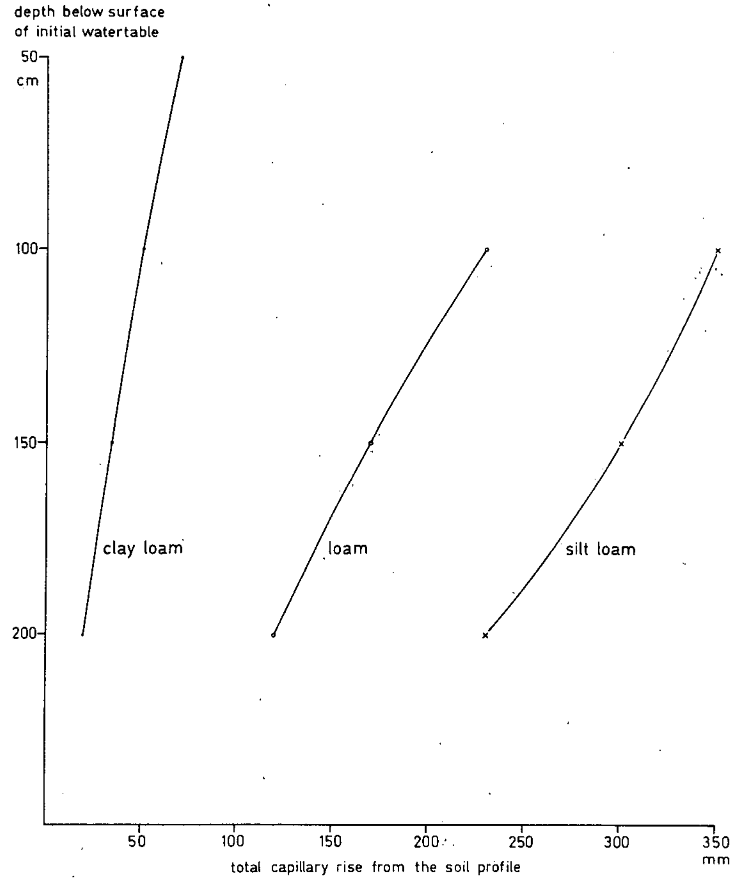
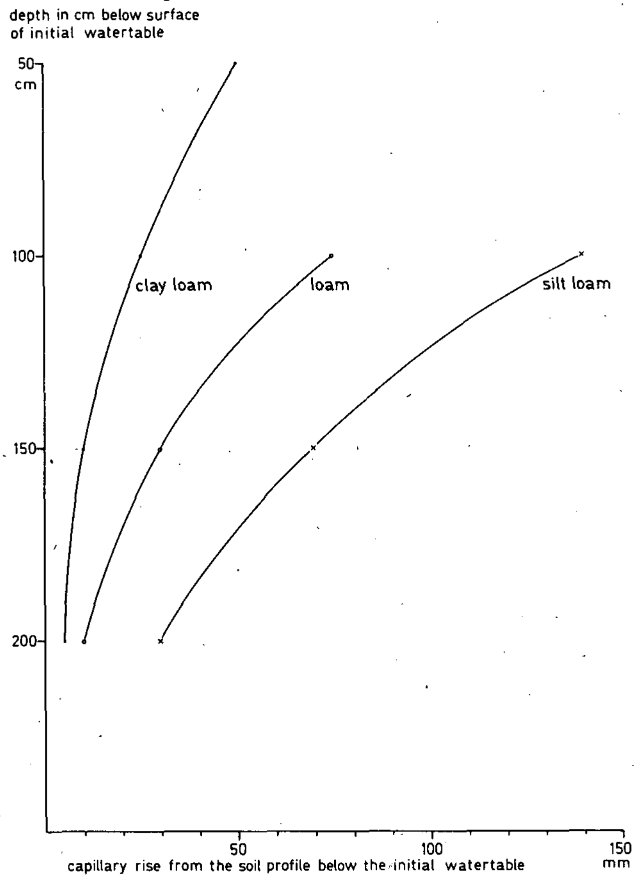


Fig. 4 Relation between capillary rise and depth of initial watertable in the case of a falling watertable during a six-month fallow period.



As in Section 2, a clear difference appears between the soil types, the silt loam again yielding the highest values for the amount of water displaced by capillary flow. The deeper the initial watertable, the smaller the amount of capillary rise and moreover the smaller the percentage of moisture originating from the soil profile below the initial watertable. Since in practice this moisture may have a higher salt concentration than the moisture provi-

Fig. 5 Relation between capillary rise and depth of initial watertable in the case of a falling watertable during a six-month fallow period.



ded by the soil profile above the initial watertable, a low initial watertable at the start of the fallow period means less capillary rise and a smaller supply of salts. The lower the initial watertable, the smaller will be the fall of the watertable during the fallow period, as is shown in Fig. 6, which also illustrates that the fall decreases with time.

In reality, however, the fall of the watertable and the amount of capillary rise will be less than the values in Table 1, because of the development of a surface mulch. Figure 7 shows that where a surface mulch exists, this layer will have a lower moisture content, whereas below the surface mulch the moisture content will be higher.

Fig. 6 Fall of the watertable during a six-month fallow period.

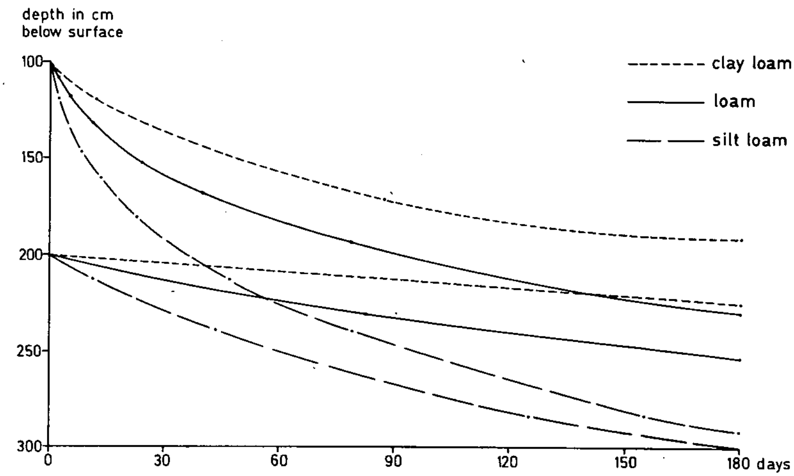
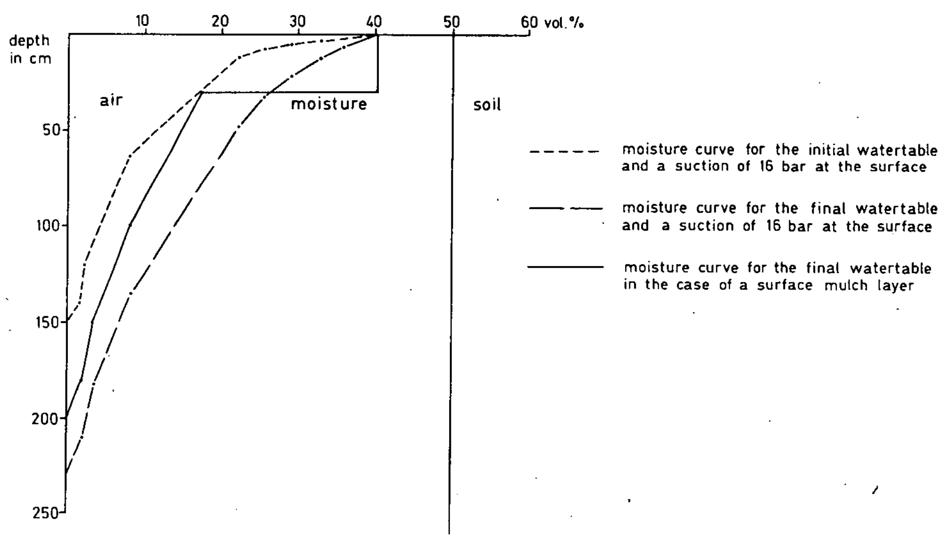


Fig. 7 Moisture displaced by capillary rise in the case of a falling watertable with and without a mulch layer.



The development of a surface mulch depends on the local conditions of soil type, climate, and crop:

- a winter crop depleting the moisture content of the rootzone favours the development of a mulch layer to a considerable depth,
- tillage of the surface layer has a similar effect,
- high evaporation exceeding the maximum capillary flow velocity is favourable for developing a surface mulch, whereas an evaporation rate matching the maximum capillary flow velocity tends to cause a gradual increase of the suction towards the surface and is unfavourable for the development of a surface mulch,
- summer rainfall just wetting the soil and increasing its capillary conductivity, but not providing percolation for leaching, also has an unfavourable effect.

The fall of the watertable and amount of capillary rise will vary for the same soil type according to local conditions. If capillary rise is not fed by seepage, the fall of the watertable will decrease to almost zero since capillary conductivity of the soil profile decreases with increasing depth of the watertable and moreover a surface mulch generally increases with time. The watertable level occurring during the fallow period at which capillary rise is reduced to almost zero, can be defined as the critical depth.

By determining the chloride content of the soil profile, it can easily be checked whether a constant watertable during a fallow period is due to the absence of capillary flow or to capillary flow whose rate equals the seepage rate.

The following conclusions can be drawn:

With a falling watertable, the amount of capillary rise originating from the soil profile below the initial watertable at the start of the fallow period will in general be small and can be reduced by lowering the initial water table and by creating a mulch layer.

Where no seepage occurs, the watertable will fall to a stable level at which capillary rise is reduced to almost zero and which can be defined as the critical depth for a certain soil under local conditions.

TABLE 2.

The height of capillary rise (Z) in relation to flow velocity (v), suction (ψ), and soil moisture content (θ).

		Clay loam, pore volume 44,5%							
	v(cm.day^{-1})	0.5	0.4	0.3	0.2	0.15	0.1	0.06	0.02
ψ (cm)	θ (vol.%)	Z (cm)							
- 20	42.4	12.1	12.1	14.3	15.8	16.7	17.7	18.5	19.5
50	41.5	25.6	28.3	31.7	36.0	33.6	41.7	44.7	48.1
100	40.6	37.6	42.4	48.8	57.7	63.8	71.8	80.3	92.0
250	38.5	43.6	49.7	58.2	71.2	80.9	95.2	113.7	153.9
500	36.5	43.9	50.1	58.7	71.8	81.8	96.4	115.8	160.2
1 000	34.4	44.0	50.3	58.9	72.2	82.3	97.2	117.1	164.0
2 500	32.0	44.1	50.4	59.1	72.5	82.7	97.9	118.2	167.2
5 000	28.6	44.2	50.5	59.3	72.7	83.0	98.2	118.8	169.1
10 000	26.5	44.3	50.6	59.4	72.9	83.2	98.5	119.4	170.7
16 000	24.2	44.3	50.6	39.4	73.0	83.3	98.7	119.6	171.5
		Loam, pore volume 50.3%							
20	48.3	17.7	18.2	18.6	19.0	19.3	19.5	19.7	19.9
50	46.7	42.2	43.5	45.0	46.5	47.3	48.2	48.9	49.6
100	42.0	74.0	77.7	82.1	87.0	89.8	92.0	95.6	98.5
250	28.1	102.6	111.0	122.1	137.8	148.8	164.0	182.0	214.3
500	24.8	104.6	113.7	125.7	143.1	155.7	174.2	198.4	256.5
1 000	21.3	105.9	115.3	127.8	146.3	160.1	180.6	209.1	287.2
2 000	16.7	107.0	116.7	129.7	149.1	163.8	186.2	218.3	314.4
5 000	14.2	107.7	117.5	130.7	150.7	165.9	189.4	223.7	330.5
10 000	11.6	108.2	118.2	131.6	152.7	167.7	192.0	228.0	343.5
16 000	9.8	108.5	118.5	132.1	152.7	168.6	193.4	230.3	350.5
		Silt loam, pore volume 50.9%							
20	48.7	18.3	18.6	18.9	19.3	19.4	19.6	19.8	19.9
50	47.4	44.2	45.3	46.3	47.5	48.1	48.7	49.2	49.7
100	46.1	81.2	84.2	87.6	91.3	93.3	95.4	97.2	99.0
250	32.5	127.7	137.2	149.2	165.7	176.8	191.3	207.3	231.3
500	27.9	134.4	145.4	160.0	181.4	197.2	220.5	252.1	326.6
1 000	20.5	138.6	150.7	167.0	191.9	211.1	241.0	265.4	414.7
2 500	13.7	142.3	155.3	173.0	201.0	223.2	259.1	315.2	500.3
5 000	12.5	144.4	157.9	176.6	206.3	230.2	269.6	322.8	552.2
10 000	10.3	146.1	160.1	179.4	210.6	236.0	278.2	347.0	594.7
16 000	9.2	147.0	161.2	180.9	212.8	329.0	282.8	354.6	617.4

4. Use of critical depth for determining drain depth

Only where seepage occurs can capillary flow lead to salinization. If seepage flow cannot be intercepted or eliminated, e.g. by lining leaky irrigation canals or by reducing irrigation losses in adjacent areas, the concept of the critical watertable depth, as defined in Section 3, can be used for determining the drain depth.

To minimize the capillary rise of seepage water, the drain depth should be calculated as the sum of the critical depth and the hydraulic head necessary to discharge the seepage flow to the drains (Figure 8a).

Since the capillary conductivity of a silt loam is higher than that of a clay loam, the critical depth in a silt loam will be lower. So a correlation will be found between the critical depth, defined as the depth at which ca-

Fig. 8a Drain depth and critical watertable depth in the case of seepage.

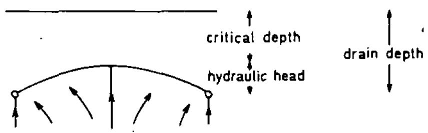


Fig. 8b Discharge of percolation and seepage water.

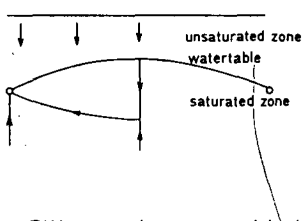
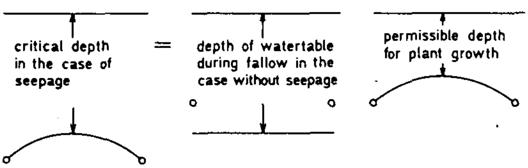


Fig. 8c Difference between critical depth and permissible depth of watertable.



pillary flow reduces to 1 mm/day for a suction gradually increasing to 16 bar at the surface and the critical depth defined as the watertable during a fallow period at which capillary flow reduces to almost zero. Under certain conditions the values may even be the same. This does not mean however, that under those conditions capillary flow will equal 1 mm/day, since in reality the presence of a surface mulch tends to reduce capillary flow to much lower values.

Where percolation water and seepage water are discharged together, no risk of capillary rise exists, since the percolation water is moving downward in the unsaturated zone and floating on seepage water in the saturated zone (Fig.8b). However, since discharge of percolation water is rarely a continuous process, whereas seepage is, one should always determine the drain depth as the sum of critical depth and hydraulic head necessary for discharging seepage flow.

In the absence of seepage drain depth does not depend upon the critical watertable depth but upon the permissible depth of the watertable depth for plant growth. This depth will in general be shallower than the critical depth, at least for annual crops.

The difference in drain depth between a situation with seepage and one without, as well as the difference between the critical watertable depth for capillary flow and the permissible depth for plant growth is shown in Fig.8c.

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Paper 4.07

EXTENDED SINGULAR DRAINAGE: AN INTERESTING LAYOUT FOR DRAINING IRRIGATED LANDS

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Summary

The singular drainage layout with laterals of a length not exceeding 250 to 300 m (see Fig.1A) is the commonly used system in flat areas. The system requires a relatively long system of open drains which have to be deep in irrigated areas and are as a consequence expensive in terms of construction, maintenance and loss of land. Open drains can be avoided by the composite drainage layout (see Fig.1B), which system has other disadvantages, however. Attention is drawn to another alternative, viz. the extended singular layout (see Fig.1C), which like the composite system does not use open drains and offers the advantages of the singular layout.

1. Singular and composite drainage

The singular system (see Fig.1A) is the traditional method of drainage of flat lands in humid areas. The system consists of pipe drains with a length of 100 to 250 m flowing into open collector drains. The maximum length of the pipe drains is related to the required slope and/or the length which can be cleaned with a flushing apparatus.

The main advantages of the singular system are: simple layout and construction, easy check on proper functioning and easy maintenance of the pipe drains.

The layout is characterized by the vast length of the collector ditches. In irrigated areas the pipe drainage and thus also the ditches have to be deep which implies wide cross-sections, resulting in high construction and maintenance costs. Moreover, structures are needed for the crossing of ditches with the irrigation and road systems. These and other disadvantages of open drain ditches in irrigated areas can be summarized as follows:

- high construction and maintenance costs;
- loss of valuable irrigable land;
- constraints on optimum layout of irrigation, road and farming fields.

In particular, proper maintenance of deep open drains often appears to be much more difficult than was expected in the planning phase. The effect of drain depth on land loss and excavation volume and hence, on construction costs, is demonstrated in Table 1. The figures presented in the table refer to ditches at 500 m intervals, with a bottom width of 1.0 m and a bed level that is 0.6 m below the pipe drain level, a spoil bank of maximum 1.5 m height at one side, and 2 m berm, and 3:2 side slopes of ditch and spoil bank.

TABLE 1. Land loss and excavation in relation to drain depth

Depth (m)		Excavation (m^3/m)	Land loss (%)
pipe drain	open drain		
1.0	1.6	5.4	2.7
1.5	2.1	8.7	3.5
2.0	2.6	12.7	4.3
2.5	3.1	17.5	5.2

In the composite drainage system as sketched in Fig.1B, the lateral drains discharge into a buried collector pipe. This system avoids the disadvantages of the open drains and, if properly constructed, it is fully acceptable from a drainage point of view. The system is, however, not widely used as it has also certain disadvantages such as:

Construction problems in silty or sandy soils with high groundwater tables. The installation of the collector pipe under these conditions may be an extremely difficult operation as the special drainage machines are able to install large-diameter collector pipe at great depths in these soils will, as a rule, not be available. Other installation methods requiring difficult and expensive dewatering techniques have then to be used.

- Loss of hydraulic head in the junction of lateral and collector, which means a greater depth of collector- and main system than in the comparable singular layout.
- Maintenance and inspection of the laterals is more difficult than in the singular layout.

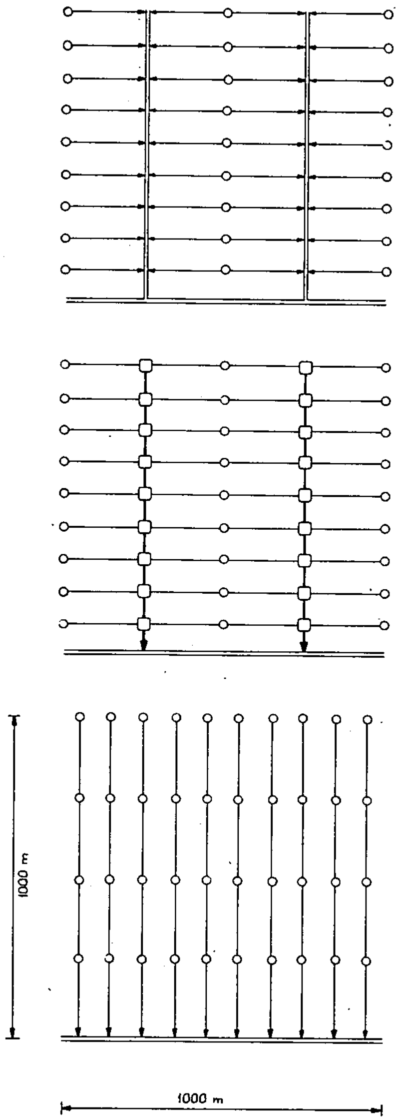

FIG. 1
ALTERNATIVE DRAINAGE LAY-OUTS

A. SINGULAR SYSTEM,
LATERALS OF 250 m

B. COMPOSITE SYSTEM,
LATERALS OF 250 m

C. EXTENDED SINGULAR SYSTEM,
LATERALS OF 1000 m

LEGEND

OPEN DRAIN
COVERED COLLECTOR DRAIN WITH JUNCTION MAN HOLE
LATERAL DRAIN
MAINTENANCE MAN HOLE
OUTFLOW STRUCTURE

2. Extended singular drainage

The extended singular drainage system aims, just like the composite system, at reducing the length of open ditches as much as possible, but it tries at the same time to maintain most of the advantages of the singular lay-out.

The system is a singular layout with laterals of great length discharging directly into the main drainage channel, which eliminates the need for open or closed collector drains (Fig.1C).

The extended layout is, in particular, suitable for large, flat, regularly sloping areas. Design poses no special problems. Natural slope is to be used wherever possible. In horizontal areas, drains have to be designed with a very small gradient to avoid too great a difference in drain depth between the upper and lower drain ends; slight slopes call for the use of a large-diameter drain pipe, which has a cost-increasing effect. The length of the extended laterals is limited by topography and by the requirements as regards minimum slope, maximum available diameters and by cost factors.

3. Cost aspects

Analyses of construction and maintenance costs have to show which system is to be preferred economically in a given situation. Examples of construction and maintenance cost analyses are given in Tables 2 and 5. The analyses have been based on the layouts shown in Figure 1 and on the design data and unit prices as presented in Tables 3 and 4, respectively. Table 2 shows for the given example that construction costs of the extended layout are least expensive without even accounting for the land loss in the singular layout. The difference of costs with the composite layout is small.

Analysis of the maintenance costs is difficult. The yearly maintenance expenses depend on many factors such as the stability of the side slopes and the depth of the drain ditches and the risk of silting up of pipe drains in a given soil. In Table 5 it has been attempted to compare the maintenance costs for the different systems.

The result shows that in the conditions as assumed the maintenance costs for three systems do not differ much if the laterals are cleaned each year. For the two- and five-year lateral cleaning programmes, the singular system is most expensive and the extended one least expensive.

Yearly maintenance of laterals is usually not required. The common practice is to clean them every two to five years. Furthermore, 3% of the investment costs of maintenance of the collector ditch is an average value. Under certain conditions, that is in soils containing sandy layers, the cost of proper maintenance may be as high as 5 to 10% of the construction costs.

4. Conclusion

The extended singular system is an attractive layout worthwhile to be considered for large-scale drainage in flat irrigated areas. It may be favourable in terms of construction and maintenance costs, but it offers, moreover, the advantages of simple design, simple construction, easy check on proper functioning, easy maintenance and no constraints on the optimum layout of the irrigation and road systems, and of the farming fields.

An extended singular layout in flat areas implies the need for the application of drain slopes that are as gentle as 0.5% or less. This is not felt to be a serious disadvantage, however.

TABLE 2. Investment costs in US $ per 100 ha (1977)

Tentative cost estimates for comparison purposes	Unit	Unit costs US $	Spacing 50 m quantity	Spacing 50 m costs	Spacing 100 m quantity	Spacing 100 m costs
		see Tab. 4 & 5				
S i n g u l a r d r a i n a g e						
Field drain, corrugated pvc, diameter 80 mm	m	4.80	20,000	96,000	10,000	48,000
Collector ditch-2,000 m, 16.5 m^3/m	m^3	2.80	33,000	92,400	33,000	92,400
Manholes for maintenance	piece	3.60	40	14,400	20	7,200
Outlet structures	piece	1.20	80	6,400	40	3,200
				209,200		150,800
				100%		100%
C o m p o s e d d r a i n a g e						
Field drain, corrugated pvc, diameter 80 mm	m	4.80	20,000	96,000		
Field drain, corrugated pvc, diameter 100 mm	m	5.80		10,000		58,000
Collector pipe, concrete, inside dia. 300 mm	m	20.00	2,00	40,000	2,000	40,000
Manholes for maintenance	piece	360	40	14,400	20	7,200
Junction manholes for maintenance	piece	440	40	17,600	20	8,800
Heavy outlet structures	piece	200	2	400	2	400
				168,400		114,400
				80%		80%
E x t e n d e d s i n g u l a r d r a i n						
Field drain, corrugated pvc, diameter 100 mm	m	5.80	10,000	58,000		
Field drain, corrugated pvc, diameter 125 mm	m	8.00	10,000	80,000	5,000	40,000
Field drain, corrugated pvc, diameter 160 mm	m	10.4			5,000	52,000
Manholes for maintenance	piece	360	80	28,800	40	14,400
Outlet structures	piece	80	20	1,600	10	1,600
				168,400		108,000
				80%		72%

TABLE 3. Basic design data

Singular system; q = 3 mm/day = 0.35 l/s/ha

design water level, branch drain	2.6 m below soil surface
design water level, ditch collector	2.5 m below soil surface
lower end, lateral drain	2.4 m below soil surface
upper end, lateral drain	2.0 m below soil surface

slope 1.6 m per 1,000 m; required inside/outside diameter corrugated drain

pipe for spacing 50 m: 47/80 mm and for spacing 100 m: 61/80 mm;

section collector drain: bottom 1 m, depth 3 m, side slopes 2:1, excavation 16.5 m^3/m, land loss 5%.

Composed system; q = 3 mm/day

design water level, branch drain	2.6 m below soil surface
lower end, subsurface collector	2.5 m below soil surface
upper end, subsurface collector	2.2 m below soil surface
slope of subsurface collector	0.3 m per 1,000 m
upper end of lateral drain	2.0 m below soil surface
slope of lateral drain	0.5 m per 1,000 m
required diameter collector drain	300 mm (inside)

required inside/outside diamater corrugated lateral drain:
at spacing of 50 m: 59/80 mm; at spacing of 100 m: 76/100 mm

Extended system; q = 3 mm/day

design water level, branch drain	2.6 m below soil surface
lower end, extended lateral drain	2.5 m below soil surface
upper end, extended lateral drain	2.0 m below soil surface
slope of extended lateral drain	0.5 m per 1,000 m

required inside/outside diameter corrugated drain pipe:
at spacing of 50 m, upper 500 m: 76/100 mm, lower 500 m: 98/125 mm
at spacing of 100 m, upper 500 m:98/125 mm, lower 500 m:128/160 mm

TABLE 4. Unit prices (adopted)

Lateral drain, corrugated PVC (US $ per m')

outside diameter (mm)	80	100	125	160
pipe	1.00	1.60	2.80	4.80
filter	2.00	2.40	2.80	3.20
laying cost	1.80	1.80	2.40	2.40
TOTAL	4.80	5.80	8.00	10.40

Collector drain, concrete, diameter 300 mm (US $ per m')

pipe	10.0
laying	8.0
other costs	2.0
TOTAL	20.0

TABLE 5. Maintenance costs in US $ per year 1977

Frequency of cleaning of laterals	each year		every 2 years		every 5 years	
spacing of laterals	50 m	100 m	50 m	100 m	50 m	100 m
Singular system						
Collector ditch-3% year	2,770	2,770	2,770	2,770	2,770	2,770
Structures-3%/year	620	310	620	310	620	310
Laterals ϕ 80	5,200	2,600	2,600	1,300	1,040	520
TOTAL	8,590	5,680	5,990	4,380	4,430	3,600
Composed system						
Collector drain-0.150/m	1,200	1,200	1,200	1,200	1,200	1,200
Structures-3%/year	970	490	970	490	970	490
Laterals ϕ 80	6,400		3,200		1,280	
Laterals ϕ 100		3,600		1,800		720
TOTAL	8,570	5,290	5,370	3,490	3,450	2,410
Extended singular system						
Structures-3% /year	910	480	910	480	910	480
Laterals ϕ 100	3,600		1,800		720	
Laterals ϕ 125	4,000	2,000	2,000	1,000	800	400
Laterals ϕ 160		2,200		1,120		440
TOTAL	8,510	4,680	4,710	2,600	2,430	1,320

Unit costs for cleaning of laterals in US $ per meter

Outside diameter (mm)	Singular layout	Composed and extended layout
80	0.20	0.24
100	-	0.27
125	-	0.30
160	-	0.33

Paper 4.08

DRAINAGE IN LAND RECLAMATION IN THE USSR

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Summary

The main idea of this article is to give general information on the modern situation and some principal ways of development in agricultural drainage in the USSR.
A very important annual amount of drained lands (about 1 million hectares) during last years is obtained in the USSR on the basis of the results of scientific researches and application of modern technique and technology. It assures a high level of return of investments in drained lands (not more than 6-7 years at the best farms).
At the same time there are some scientific and technical problems to be solved in different climatic and geographical regions with intensive development of drainage.

Introduction

Drainage is one of two most important ways of land reclamation in the USSR. The importance of drainage is determined by the distribution of agricultural land according to the degree of natural moisture and by the total potential reclamation area in the zone of sufficient moistering. Early in 1973 the total agricultural area was about 608 mln ha, including more than 225 mln ha of arable land. According to the degree of natural moisture the distribution of agricultural land is as follows: the zone with sufficient moistening (precipitation greater than 700 mm) - 1%, the zone of insufficient moistening (precipitation 400-700 mm) - 59% and the arid zone (with precipitation less than 400 mm) - 40%. The zone of sufficient moistening is relatively small (1%) but over 40 per cent of the value of the agricultural gross output is produced in this zone. Moreover this zone is notable for its high developed industry and the great number of towns and other localities. The most important regions of the zone are: the north-western and central parts of the Russian Federation, the Transurals, the Baltic republics, Byelorussia, the western and northern parts of the Ukraine, Kolkhida (Caucasus) and some areas in the Far East.

Before discussing the potential area for drainage it is necessary to say that the drained area is already important in the USSR. At the beginning of 1974 there were over 12 mln ha of drained land including the area with closed drainage of about 4.9 mln ha. But the potential area for drainage is much larger and amounts to approximately 92 mln ha.

According to the general agricultural policy the Soviet Government adopted in 1966 a very important decision of development of land reclamation in the USSR and in 1974 another important decision of land reclamation in the non-chernozem zone in the European party of the USSR was taken. Therefore in the period 1976-1980, 4.7 mln ha of land must be drained and in the period 1975-90 approximately 9-10 mln ha in the non-chernozem zone including 7-8 mln ha by means of closed drainage.

Closed and open drainage

Closed drainage is applied in many regions of the USSR, practically on all new drained areas. Open drainage is applied in areas difficult for utilisation of drain layers. In 1965 closed drainage construction in the USSR was 1.96 mln ha, in 1970 - 3.29 mln ha and in 1975 - 5.9 mln ha. Independent plots drained by open canals often have the dimensions of 10-15 ha, but in the USSR closed drainage gives possibility of increasing the dimensions of such plots to 100 ha. There are already a few lots with closed drainage with the dimensions of 50-80 ha.

Drainage on irrigated lands

Modern irrigated lands are usually exploited together with drainage systems. In the regions with existing traditional irrigation only open collector systems are generally constructed. But new advanced irrigation systems are constructed mainly with subsurface drainage. This drainage (horizontal) is laid at a depth of 3.0 - 3.5 m and deeper with specific lengths of 25-100 m/ha. Often the horizontal drainage is constructed in combination with vertical drainage and also used in the traditional zones of irrigation. The depth of the wells is different, as a rule 50-70 m but in some cases to 100 m. The density of salinity control wells is one well for 70-120 ha if the drop of the groundwater level is 1.0 - 1.5 m.

During some years a method of vacuum composite drainage has also been studied in the USSR.

To intensify the movement of the saline water from the lower levels to the drainage pipes the intensifying wells are constructed. The radius of influence of such a well is about 70-120 m in the case of horizontal drainage laid at a depth of 2.5 m and about 180-230 m with drainage laid at 4.2 m.

Polder systems

Polder systems are also on reclaimed lands in the USSR. The dimensions of such systems are from several hundred hectares to several thousand hectares. The area of land suitable for drainage with polder systems is very important. Only in the non-chernozem zone is it about 2 mln ha.

Technical characteristics of drainage systems

The main method of construction of drainage systems in the USSR is a uniform distribution of regular tubular drainage over an area. Drains are mostly formed of potter pipes (70 per cent in all drained lands) with spacing of 10-50 m (usually 20-25 m). In peaty and mineral soils round potter pipes with inner diameter of 5-10 cm are employed (length of pipes is 0.3 m). Employed plastic corrugated pipes have a diameter of 5-12.5 cm.

Closed collectors (of different orders) are usually constructed of potter, concrete, reinforced concrete and asbestos-cement pipes with a diameter of 30-100 cm. In the future plastic collector pipes with a diameter of 60 cm will be employed.

In drainage systems different filter materials are used: sand-gravel mixtures, moss, peat dust, mineral wool, glass fibre mats.

Some ways of development of drainage systems

The vast program of drainage works in the USSR (the average annual drained area is about 1 mln ha) requires a solution of some problems important from technical and economic point of view. They are:

- improvement of techniques and technology of drainage of heavy mineral soils including combined techniques with application of chemical products;
- application of capsul-type pumps in pumping stations;
- development of axial monoblock electric immersible pumps with a capacity of 250-400 cubic metres per hour and electric immersible capsule-type pumps with a capacity of 600-3200 cubic metres per hour.

Effectiveness of drained lands

The drained lands in the USSR are mostly situated in the regions with relatively temperate and continental climate. But even under such conditions the yield obtained from these lands is twice or three times more than from non-reclaimed lands. At the best farms the investments in land reclamation (drainage, stubbing, removal of rocks, levelling and other works) are returned in not more than 6-7 years. During the last five-year plans (1966-1970 and 1971-1975) the state investments in drainage works were accordingly 28.8 and 26.9 per cent of all investments in land improvement but in absolute figures they were double those figures.

References

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Paper 4.09

DESIGN OF GRAVEL ENVELOPE FOR SILTY AND FINE SANDY SOILS IN PAKISTAN

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Summary

This paper deals with the design of a gravel filter for a pilot area in Pakistan. The criteria are applied to actual soil conditions, from which a range of required grading of the filter results. From available filter material in Pakistan, gravel from Shadi Shaheed is selected as most suitable, on the basis of grading, distance from quarry to pilot area, and performance in a sand tank model. Two aspects of the execution are discussed: contamination of gravel and piping.

1. Introduction

When designing a tile drainage system, one has to decide whether an envelope around the pipes will be necessary or not. This decision and, if required, the choice of an envelope of suitable type and characteristics is of vital importance and requires due attention. The dominant factor in this respect is the condition of the soil, in which the drains are to be installed.

In general, tile drains in a stable clay soil of good permeability do not require an envelope, whereas the installation of pipes in a silty soil without a protective filter is likely to lead to a failure. For each situation, the ultimate selection of the envelope material depends on the stability, the texture, and the permeability of the base material in the vicinity of the drain.

Two main functions of the envelope may be distinguished:

a) The function as a filter, to prevent soil particles from entering the drain.

b) The function as a highly permeable surround, to facilitate the flow of water towards and into the drain pipe.

The first function requires rather fine pores, while the second function can only be performed well, if the material has sufficient permeability, i.e. relatively large pores. The design of the envelope aims at the best compromise between these two contrary requirements.

Various types of material are in use as filters. In the Western United States and in many Middle Eastern countries, sand and gravel filters are applied, although the cost of transport and handling of this bulky material is considerable. In most European countries, organic and synthetic protective filters are used.

The Drainage and Reclamation Institute of Pakistan in Hyderabad studying the possibilities to introduce tile drainage in salinized and waterlogged areas, decided to set up an experimental field. The present paper deals with the design of a gravel filter for this pilot area in Khairpur-East, a few hundred kilometers north of Hyderabad, according to criteria available from literature and taking into account the soil conditions in the area.

Because of the soil conditions, the selection of a suitable gravel is not easy. Soils are predominantly silt loams, underlain by silt-containing fine sandy and sandy layers, which in most places have been found to commence within drain depth. These light-textured subsoils, in combination with a groundwater table of 2 to 3 feet below ground level, give the profile a very unstable character. These sloughy conditions aggravate the problems of the mechanical installation of the drains and filter material.

Obviously, the technical aspects alone cannot provide the basis for a justified selection of a suitable filter material. The economical aspects are very important. As mentioned above, transport and handling cost of gravel is considerable. This aspect is an important consideration in the ultimate selection, as these costs can form a substantial part of the total cost of drainage.

2. Criteria for the design of gravel envelopes

Since the 1920's, criteria for the design of filters have been studied by civil engineers. Criteria were first formulated by Terzaghi (1921). In the beginning, the same criteria were used for land drainage, as no other criteria were available. Later, criteria have been adjusted for land drainage purposes. In the 1940's and 1950's, much research was done on filter design. Recently, Spalding, of the UK Road Research Laboratory, studied the many criteria which had been formulated (SPALDING,1970). He selected the criteria of the US Waterways Experiment Station, combined with those of the US Corps of Engineers as the most reliable ones. Other criteria were suggested simultaneously by the US Soil Conservation Service (USDA Soil Conservation Service, 1971) and by the US Bureau of Reclamation (WINGER and RYAN, 1970). In the following chapters, these criteria will, wherever convenient, be referred to as RRL-, SCS-, and USBR-criteria, respectively.

2.1 UK Road Research Laboratory criteria

Some of the criteria to be applied depend on the shape of the grading curve of the soil. This shape is characterized by the ratio: D60S/D10S.* If D60S/D10S is smaller than or equal to 1.5 the soil is called uniform. A uniform soil consists of well-sorted material, as occurs in dune sand. The opposite of a uniform soil is a well-graded soil, containing particles of many different sizes. A soil is called well-graded if D60S/D10S is greater than or equal to 4.0.

Spalding recommends the following general criteria:

for filtration	1.	D15F $\leqslant$ 5 D85S.*
	2.	D15F $\leqslant$ 20 D15S
	3.	D50F $\leqslant$ 25 D50S
for permeability	4.	D15F $\geqslant$ 5 D15S

Only, for a uniform soil (D60S/D10S $\leqslant$ 1.5) criterion 1 changes into:

1.* D15F $\leqslant$ 6 D85S

and for a well-graded soil (D60S/D10S $\geqslant$ 4.0) criterion 2 changes into:

2.* D15F $\leqslant$ 40 D15S

A further general condition is that the filter should not be gap-graded, the grading curve should be a smooth line.

**NOTE:* *S stands for soil, F stands for filter*

For the coarser fraction of the filter material, a relation between the dimensions of the perforations of the drain and the D85F value can be used to add another criterion. Spalding mentions that, in order to prevent that particles of the filter material will be washed into the pipe through the perforations, the maximum slot width should be smaller than or equal to 0.83 D85F. If the slot width is fixed, the following criterion for the filter results:

$$5. \quad D85F \geqslant \frac{\text{slot width}}{0.83}$$

Fig.1 shows an example of the application of these criteria for a soil with the following characteristics:

D10S	=	0.020 mm
D15S	=	0.022 mm
D50S	=	0.035 mm
D60S	=	0.040 mm
D85S	=	0.087 mm

The D60S/D10S ratio is 0.040/0.020 = 2, which means that the soil is not uniform, nor well-graded, and hence criteria 1 through 5 should be applied.

1.	D15F	$\leqslant 5$	$\times$	0.087	=	0.44 mm
2.	D15F	$\leqslant 20$	$\times$	0.022	=	0.44 mm
3.	D50F	$\leqslant 25$	$\times$	0.035	=	0.88 mm
4.	D15F	$\geqslant 5$	$\times$	0.022	=	0.11 mm
5.	D85F	$\geqslant 1.5/0.83$			=	1.8 mm

Combination of these criteria gives:

from 1, 2, 4:	$0.11 \leqslant$ D15F $\leqslant 0.44$ mm
from 3:	D50F $\leqslant 0.88$ mm
from 5:	D85F $\geqslant 1.8$ mm

The smooth curves marked RRL, drawn in Fig.1 represent the approximate upper and lower limits of the filter grading curves consistent with the above calculated criteria.

2.2 US Soil Conservation Service criteria

Based on research of the Bureau of Reclamation and the US Corps of Engineers, the SCS (USDA Soil Conserv.Service,1971) recommends the following criteria:

1. $12 \leqslant D50F/D50S \leqslant 58$
2. $12 \leqslant D15F/D15S \leqslant 40$

for perforated drains:

3. $D85F \geqslant 0.5 \times$ diameter of perforations
4. $D100F < 37.5$ mm
5. $D90F < 19$ mm
6. $D10F > 0.25$ mm

Only, for uniform soils criterion 1 changes into:

1.* $5 \leqslant D50F/D50S \leqslant 10$

and a seventh criterion is added for stability:

7. $D15F/D85S \leqslant 5$

Fig.1 shows an example of the application of the SCS criteria for the same soil as mentioned in Section 2.1. The base material is not uniform, so that criteria 1 through 6 should be used.

1. $D50F \geqslant 12 \times 0.035 = 0.42$ mm
$D50F \leqslant 58 \times 0.035 = 2.0$ mm

or:

$0.42 \leqslant D50F \leqslant 2.0$ mm

2. $D15F \geqslant 12 \times 0.022 = 0.26$ mm
$D15F \leqslant 40 \times 0.022 = 0.88$ mm

or:

$0.26 \leqslant D15F \leqslant 0.88$ mm

3. $D85F \geqslant 0.5 \times 1.5 = 0.75$ mm

using the same minimum slot opening of 1.5 mm as in the preceding Sect.2.1

4. $D100F < 37.5$ mm
5. $D90F < 19$ mm
6. $D10F > 0.25$ mm

(This requirement limits the amount of fines in the filter to reduce migration of fine particles.)

The filter limits marked SCS, shown in Fig.1 indicate the range of possible filter grading curves for this specific soil according to the SCS criteria.

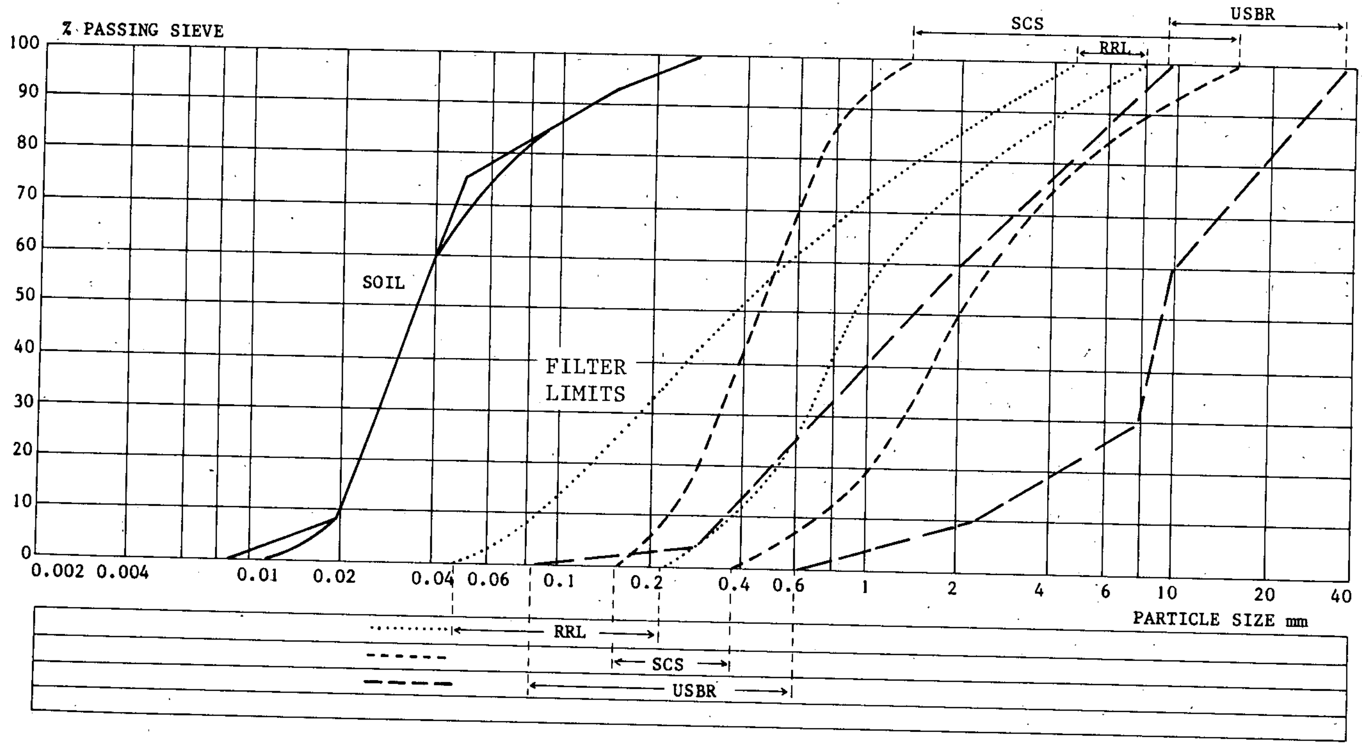

Fig.1. *Example of application of 3 sets of criteria on a soil. Curves represent upper and lower filter limits according to criteria of RRL, SCS and USBR.*

2.3 US Bureau of Reclamation criteria

Winger and Ryan (1970) are of the opinion that the criteria for protective filters, are too restrictive for agriculture and result in too expensive filters. Winger and Ryan propose more liberal criteria, thus allowing the use of cheaper filters.

In addition to the earlier mentioned ratio D60S/D10S, also called coefficient of uniformity, C_u, they use a coefficient of curvature, characterizing the shape of the gradation curve:

$$C_c = \frac{(D30)^2}{(D10)(D60)}$$

According to Winger and Ryan, any filter should be reasonably well-graded. A filter is considered well-graded, if $1 < C_c < 3$, and if $C_u > 4$ (for gravels) or $C_u > 6$ (for sands).

As D60 appears in both the C_u and the C_c, Winger and Ryan first established a relation between D60S and D60F, then they calculated D10F and D30F limits, from the formulas for C_u and C_c. The result is given in Table 1.

TABLE 1. Upper and lower limits for four base materials (in mm)

Soil	D60S	D60F	D30F	D10F
A	0.02 - 0.05	2.0 - 10.0	0.81 - 8.7	0.33 - 2.5
B	0.05 - 0.10	3.0 - 12.0	1.07 - 10.4	0.38 - 3.0
C	0.10 - 0.25	4.0 - 15.0	1.30 - 13.1	0.40 - 3.8
D	0.25 - 1.00	5.0 - 20.0	1.45 - 17.3	0.42 - 5.0

A = coarse silt loam *C = medium sand*
B = very fine sand *D = coarse sand*

In Fig.1 the grading curves of standard soil type A has been presented, together with the corresponding lower and upper filter limit curves. Some characteristic points of these filter curves, as indicated in Table 1, Col. 2, and for soil A also in Fig.1, are:

I: The values for the lower limits of D60F (2.0, 3.0, 4.0, and 5.0 mm) have been selected to provide adequate permeability of the filter material.

II: The values for the upper limits of D60F (10.0, 12.0, 15.0, and 20.0 mm) have been selected from observations of success and failure of envelopes, with regard to the movement of fines from the base material into the envelope.

III: For all four upper limit curves the permissible largest particle size has been established at 38.1 mm. It has been observed, that any coarser gravel could crack or break clay pipes, if it falls directly on the pipe from the top of the trench. Also, gravel sizes larger than 38.1 mm could cause grade and alignment problems during the construction.

IV: For the four lower limit curves the maximum size is 9.52 mm. There should be some gravel up to this size to ensure adequate permeability.

V: The fines for the upper limit curves can vary considerably, but it has been observed that, when most of the envelope consists of the larger diameter particles, part of the sand particles smaller than 0.6 mm will eventually move into the pipe. Hence the minimum size of 0.6 mm for these curves.

VI: In the lower limit curves smaller sand particles can be permitted, but obtaining adequate permeability might be a problem if more than about 5% is smaller than 0.3 mm.

VII: In the lower limit curves, grain sizes of less than 0.074 mm should not be accepted.

These seven explanatory remarks of Winger and Ryan are mainly based on practical experience of the USBR gained in the Western United States.

2.4 Comparison of the criteria

From Fig. 1 it appears that the RRL allows more fines in the filter than the SCS, whereas the USBR recommends coarser filters than the SCS. This may be understood from the fact that the three institutions developed their criteria from different backgrounds. The RRL criteria were mainly used for civil engineering works, such as road construction. This explains the relative strictness in their criteria. The SCS criteria were especially developed for agricultural drainage. They are less restrictive and place emphasis on the filter action. The USBR, observing that there was water standing above the drains, even with a carefully designed filter, concluded that emphasis in filter design should be on permeability, rather than on filter action. This has resulted in criteria which lead to coarser filters.

The difference in approach between the SCS and the USBR may find its cause in the different conditions in East and West USA. In general, the drainage projects in the Western States, where the USBR operates, are large, with long and widely spaced collectors and laterals. The large scale of these systems often requires large pipe diameters in wide trenches, which are filled with considerable quantities of gravel. This quantity of gravel keeps the base material at a safe distance from the pipe, thereby ensuring stable conditions and a low entrance resistance. This may explain the major emphasis, which the USBR places on the hydraulic function of the envelope.

In the Eastern States, the SCS is mainly working at the farm level, which implicates smaller pipe diameters, smaller trenches and a less copious use of filter material. The latter calls for more attention to the filter action of the envelope.

3. Design of a gravel filter for the pilot area

3.1 Soil conditions in the pilot area

Twelve soil samples have been collected from the pilot area in Khairpur East, taken at a depth of approximately 1.80 m below ground level. These samples were analyzed in the laboratory for their grain size distribution, using the hydrometer method and, for the coarser fractions, sieving. With two exceptions the grain size distribution curves fall within the range indicated by the two curves in the left part of Fig.2. It is a rather narrow range of well-graded soils of silty and fine sandy character. The two exceptions, not shown in Fig.3, are of a somewhat more heavy texture and may be classified as silty clay loam. This type of soil will have a much better stability than the other soils, and consequently, a drain in this soil needs less protection. The design of a filter for the pilot area will be based on the boundary distribution curves as shown in Fig.2.

The three different criteria, mentioned in Chapter 2, will be applied to a soil A, which represents the upper boundary (right curve in Fig.2), and a soil B, which forms the lower boundary (left curve in Fig.2). For these two soils, A and B, the following characteristic points can be derived (see Table 2):

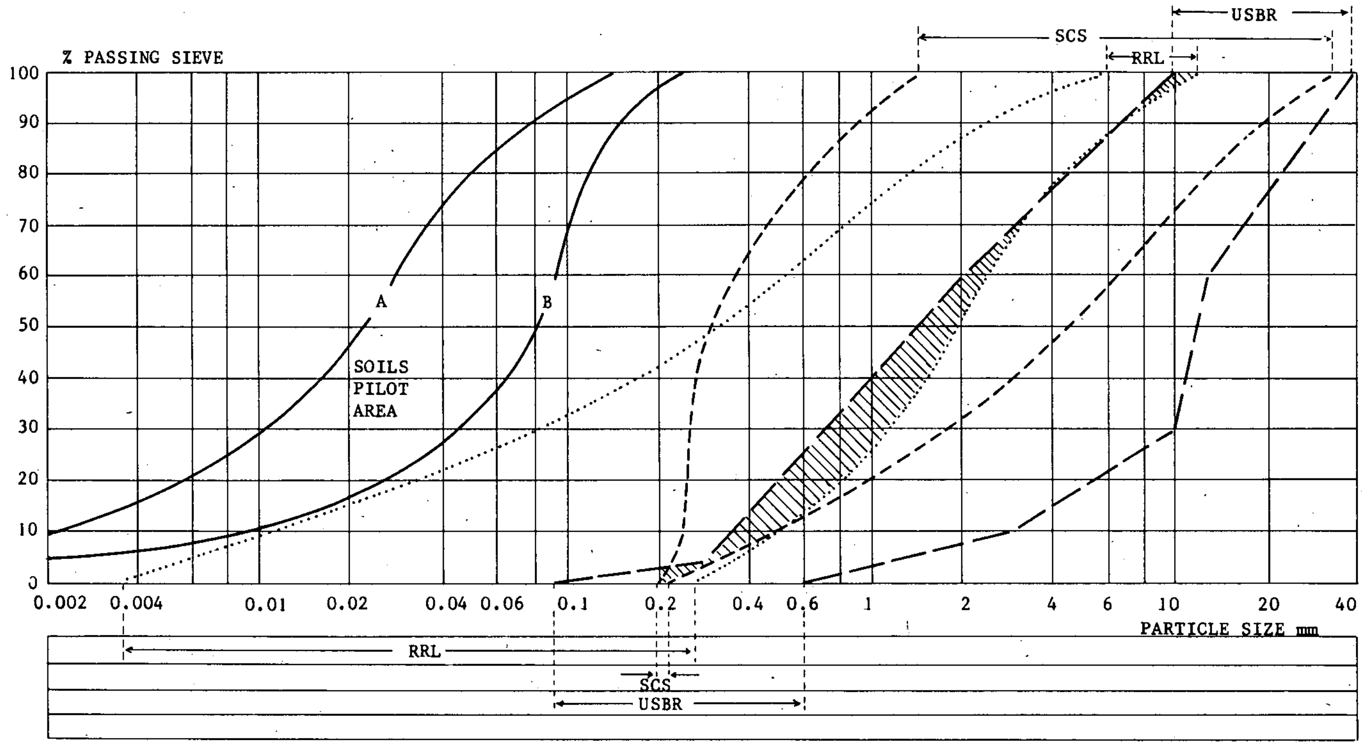

Fig.2. Combination of the uppermost and lowermost filter limits derived from the application of RRL, SCS and USBR criteria on soils in the pilot area Khairpur East (see Sect.3.2, 3.3, and 3.4).

TABLE 2. Characteristic points of distribution curves of soils A and B (in mm)

Soil	D10S	D15S	D50S	D60S	D85S
A	0.002	0.004	0.025	0.03	0.06
B	0.010	0.018	0.08	0.09	0.13

3.2 Application of the RRL-criteria

Applying the criteria mentioned in Section 2.1 to soils A and B results in:

S o i l A

D60S/D10S = 0.03/0.002 = 15

The soil is well-graded, so the criteria 1, 2*, 3 and 4 apply:

1.	$D15F \leqslant 5$ D85S	=	5×0.06	=	0.30 mm
2*.	$D15F \leqslant 40$ D15S	=	40×0.004	=	0.16 mm
3.	$D50F \leqslant 25$ D50S	=	25×0.025	=	0.63 mm
4.	$D15F \geqslant 5$ D15S	=	5×0.004	=	0.02 mm

Combination of 1, 2*, and 4 gives:

$$0.02 \leqslant D15F \leqslant 0.16 \text{ mm.}$$

and criterion 3 gives:

$$D50F \leqslant 0.63 \text{ mm}$$

S o i l B

D60S/D10S = 0.09/0.010 = 9

The soil is well-graded, so the same criteria 1, 2*, 3 and 4 apply:

1.	$D15F \leqslant 5$ D85S	=	5×0.13	=	0.65 mm
2*.	$D15F \leqslant 40$ D15S	=	40×0.018	=	0.72 mm
3.	$D50F \leqslant 25$ D50S	=	25×0.08	=	2.0 mm
4.	$D15F \geqslant 5$ D15S	=	5×0.018	=	0.09 mm

Combination of 1, 2*, and 4 gives:

$$0.09 \leqslant D15F \leqslant 0.65 \text{ mm}$$

and criterion 3 gives:

$$D50F \leqslant 2.0 \text{ mm}$$

The fifth criterion applies to both soils A and B since identical pipes (from Pak.PVC Ltd.) will be used:

5. $D85F \geqslant 1.8$ mm

According to these requirements, the upper filter limit curve for soil B and the lower filter limit curve for soil A, marked RRL, have been drawn in Fig.2. The filter should stay within the range between both filter limit curves.

3.3 Application of the SCS criteria

Applying the criteria mentioned in Sect.2.2 to soils A and B leads to:

S o i l A

The base material is graded, so the criteria 1 through 6 apply:

1. $12 \leqslant D50F/D50S \leqslant 58$

$12\ D50S = 12 \times 0.025 = 0.30$ mm
$58\ D50S = 58 \times 0.025 = 1.45$ mm $\Rightarrow 0.30 \leqslant D50F \leqslant 1.45$ mm

2. $12 \leqslant D15F/D15S \leqslant 40$

$12\ D15S = 12 \times 0.004 = 0.048$ mm
$40\ D15S = 40 \times 0.004 = 0.16$ mm $\Rightarrow 0.048 \leqslant D15F \leqslant 0.16$ mm

3. $D85F \geqslant 0.75$ mm
4. $D100F < 37.5$ mm
5. $D90F < 19$ mm
6. $D10F > 0.25$ mm

S o i l   B

The base material is graded, so the same criteria as for soil A apply:

1. $12\ D50S = 12 \times 0.08 = 0.96$ mm
   $58\ D50S = 58 \times 0.08 = 4.6$ mm $\Rightarrow 0.96 \leq D50F \leq 4.6$ mm

2. $12\ D15S = 12 \times 0.018 = 0.22$ mm
   $40\ D15S = 40 \times 0.018 = 0.72$ mm $\Rightarrow 0.22 \leq D15F \leq 0.72$ mm

Requirements 3,4,5, and 6 are the same as for soil A.Fig.2 shows the upper filter limit curve for soil B and the lower filter limit curve for soil A consistent with the above-calculated requirements. Only, for soil A the requirements 2 and 6 are mutually exclusive. Here requirement 6 overrules requirement 2.

3.4 Application of the USBR criteria

Applying the criteria mentioned in Section 2.3 to soils A and B gives:

S o i l   A

D60S = 0.03 mm

From Tab.1 it can be seen that this soil falls in USBR category A (coarse silt loam), and, consequently,the filter limits marked USBR apply,see Fig.2.

S o i l   B

D60S = 0.09 mm

From Tab.1 it can be seen that this soil falls in USBR category B (very fine sand), and, corresponding filter limits apply. In Fig.2 the upper filter limit for soil B and lower filter limits for soil A has been presented.

3.5 Discussion and recommendation

The filter limits for the pilot area, as developed from the criteria of the RRL, SCS, and USBR, have been combined in Fig.2. The curves in Fig.2

represent the extreme filter limits for each set of criteria. Only the lower limit curves for soil A and the upper limit curves for soil B have been shown.

Fig.2 demonstrates again, like Fig.1, that the coarsest filters are recommended by the USBR, whereas the RRL criteria lead to the highest percentage of fines in the filter. It is clear from Fig.2 that the ranges of the RRL filters and the USBR filters hardly have any overlap, they are almost "complementary". The upper RRL curve is approximately coinciding with the lower USBR curve. So a compromise between these two criteria should approximately be this common boundary line.

Keeping in mind the background of the three types of criteria, the SCS criteria are probably the most indicated ones for the Khairpur pilot area. Starting from this assumption, and looking only at the SCS limits in Fig.2, it can be seen that a rather wide range of filters would be acceptable, except for the amount of fines (DOF to D15F). The latter is due to the D10F restriction explained in Section 3.3. This restriction makes the shape of the SCS lower limit curve not very logical: DOF and D50F are almost identical and the filter in this range would be too uniform. For that reason, the lower limit curve at higher weight percentages than 15 may be shifted somewhat to the right hand side (taking more or less the shape of the lower RRL limit curve for 60% and above).

Shifting the lower SCS limit curve in this sense may narrow the SCS range of Fig.2 somewhat, but the range is still rather wide. It is striking, that the above-mentioned "common boundary line" of the RRL and USBR ranges lies approximately in the centre of this narrower SCS range. Therefore, it has been concluded that the final recommendation for a filter range for the Khairpur soils should approximately centre around the common overlap of all three criteria (the shaded area in Fig.2) and should spread somewhat to both sides, as shown in Fig.3. The width of this range is a compromise between the wider range to which the SCS criteria lead and the restrictions resulting from the RRL and USBR criteria.

From the approximative character of the last steps in the selection procedure it is obvious, that the upper and lower limits of the range given in Fig.3 cannot be very strict. However, a filter material should be found (or made)

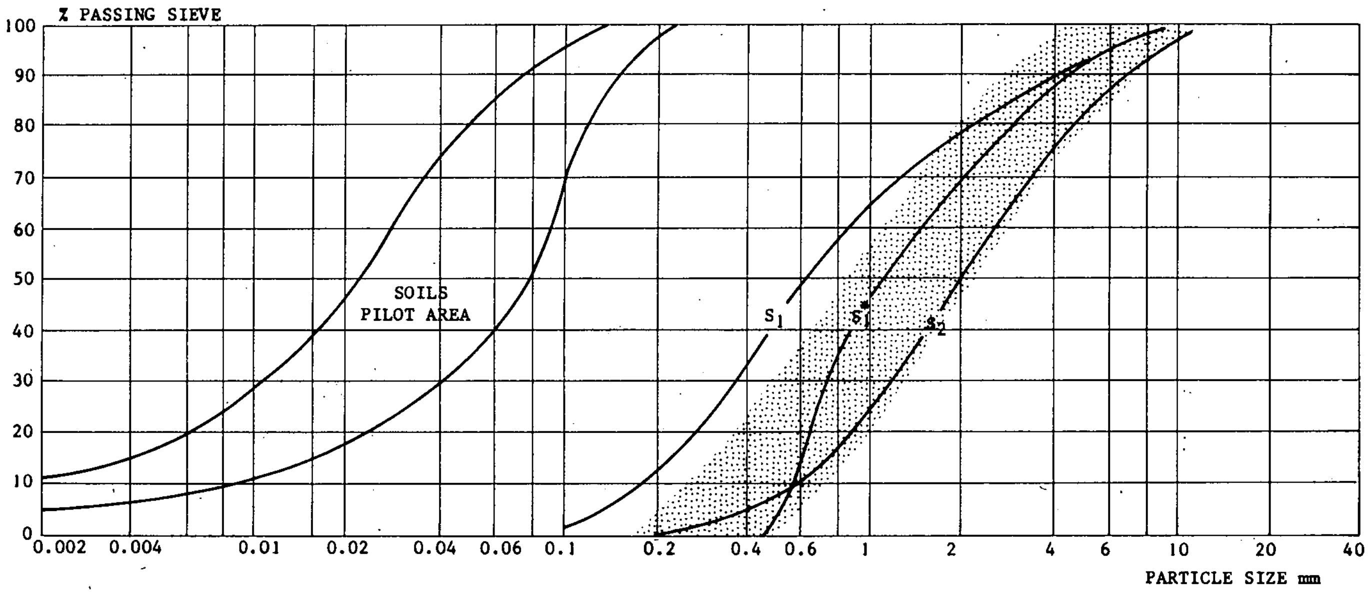

Fig.3. Particle size distribution curves of some samples of gravel collected from quarry at Shadi Shaheed.

with a grading curve in or very close to the given range, since such a filter seems to offer the best prospects for a proper functioning in the pilot area. Expressed in numbers, the particle sizes of the recommended filter material should vary from a minimum of 0.2-0.5 mm to a maximum of 5-15 mm.

4. Selection of gravel for the pilot area

Looking for a suitable filter material, gravel from several quarries has been considered. The materials were either naturally found and sieved only, or crushed first and subsequently sieved. The collected samples were sieved to determine the particle size distribution curves, which were compared with the range indicated in Fig.3.

Samples that met the requirements or were close to the required range, were tested in a sand tank model in the laboratory. Natural silica gravel from Much and Campbellpur is used as a filter for tube wells, but the quarries are located far from the pilot area, which would make these materials too expensive.

Natural limestone gravel from Bulhari, has successfully been used as a drain envelope in Gaja, near Hyderabad, but does not meet the requirements for the soils in Khairpur. Moreover the quarry is far from the pilot area.

Natural silica gravel from Umar Kot has also been used as a drain envelope in Gaja. Recent excavations have shown that the tiles enveloped in this material were clean. The lower part of the grading curve falls within the required range for Khairpur, so that it might function satisfactorily, although it is rather uniform. However this quarry is also too far from Khairpur.

Crushed limestone gravel from Kot Diji, near the pilot area, has also been considered. Some adjustments have been made to the rotating sieve in order to improve the grading, but the result was not satisfactory. After sieving, one of the samples from the dust room had a grading curve in the required range, but the sieving procedure required to obtain this material was rather laborious, and not practical.

Natural limestone gravel from a quarry at Shadi Shaheed, close to Khairpur, seemed the best material available. This gravel is mainly composed of water loose material, but quite some variation in grain size occurs within short distances.

Curves S_1 and S_2 in Fig.3 enclose a range of six samples. Curve S_1^* represents the gravel S_1 from which the fines, smaller than 0.5 mm, have been removed by sieving. As Fig.3 indicates, the Shadi Shaheed gravel fits very well in the required range. Experiments in the sand tank model with the Shadi Shaheed gravel gave good results, so that it may be concluded that this material is suitable for experiments in the pilot area in Khairpur. Some field work may be necessary to locate the best sites with a good quality of gravel in sizable pockets. Distance from the pilot area is 10 km.

5. Avoidance of contamination and piping

Contamination of the filter material with fines from the top soil should be avoided as much as possible because it would disturb the carefully selected particle size distribution. If the transport and handling of the gravel between the quarry and the drain trench would be highly mechanized, e.g.with tractors, trailers, and belt conveyors, the risk of contamination would be small. However, because of the limited scale of the pilot area in Khairpur, relatively much hand labour will be inevitable.

The gravel will be transported to the field by trucks coming directly from the quarry, and will be put in depot in the field. From the depot it will be distributed in trailers, which deposit it in heaps along the alignment of the trench, so that it may easily and quickly be taken up by the labourers to fill the gravel hoppers of the drainage machine. When the gravel is put in baskets precautions are necessary to avoid that soil will be mixed with the gravel. May be for this purpose some plastic sheets could be put on the ground underneath the gravel.

Willardson et al. (1974) found that contamination of the envelope with only 5% fine particles caused serious deterioration of the hydraulic conductivity of the drain envelope, leading to a decreased discharge rate of the drain. So, careful handling of the filter material is a necessity for the construction of a well-discharging drainage system.

Piping is the phenomenon that the first irrigation water in the newly drained land forces its way to the drains through the loosely packed trench fill, causing sedimentation of washed-out sand and silt in the porous envelope around the drain and in the drain itself. In order to diminish piping hazards, the backfill should be brought into the drain trench carefully, and should be compacted as much as is practically possible. In addition, the surface of the trench should be compacted with a tractor wheel, and the trench-fill should protrude somewhat above the ground surface. In this way the surface irrigation water is kept away from the trench as long as the backfill has not sufficiently settled.

To what extent the above-described measures will be necessary and possible in the pilot area, will depend on the actual conditions during the execution.

6. Conclusions and recommendations

1. In the Khairpur East pilot area a filter around the tile drains is necessary to prevent silting up. For the design of such a filter, criteria were considered, which had been developed and recommended by the UK-Road Research Laboratory, the US-Soil Conservation Service, and the US-Bureau of Reclamation.

2. For the silty clay loams and fine sandy soils of the pilot area the SCS criteria seemed more suitable than the others. The RRL criteria resulted in too many fines, whereas the USBR criteria led to rather coarse filters. Since the RRL and the USBR filter ranges met approximately in the centre of the SCS range, it seemed reasonable to expect that a filter in the range proposed in Fig.3 would perform satisfactorily in the difficult soils of the pilot area.

3. Gravel from quarries in Much, Campbellpur, Bulhari and Umar Kot were not suitable due to either grain size or distance from the pilot area or both.

4. The limestone gravel from the Kot Diji crusher does not have a suitable particle size distribution: it is too coarse. The production of large quantities of a suitable finer gravel would require a rather drastic change of the crusher, which does not seem feasible under the present circumstances.

5. The material in the dustroom of the Kot Diji crusher contains some fine gravel which could be sieved to a suitable gradation. For large amounts of gravel this method is not very practical.

6. At this moment, the Shadi Shaheed gravel seems to be the most promising filter material for the pilot area. It has the required grain size distribution, the quarries are very near to the pilot area, and the natural material requires little processing.

7. During transport of the gravel at site, precautions are needed to prevent contamination of the gravel with soil material, which would adversely affect the hydraulic properties of the filter.

8. To prevent piping as much as possible, careful backfilling of the trench is required, and compaction and over-filling are advisable.

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