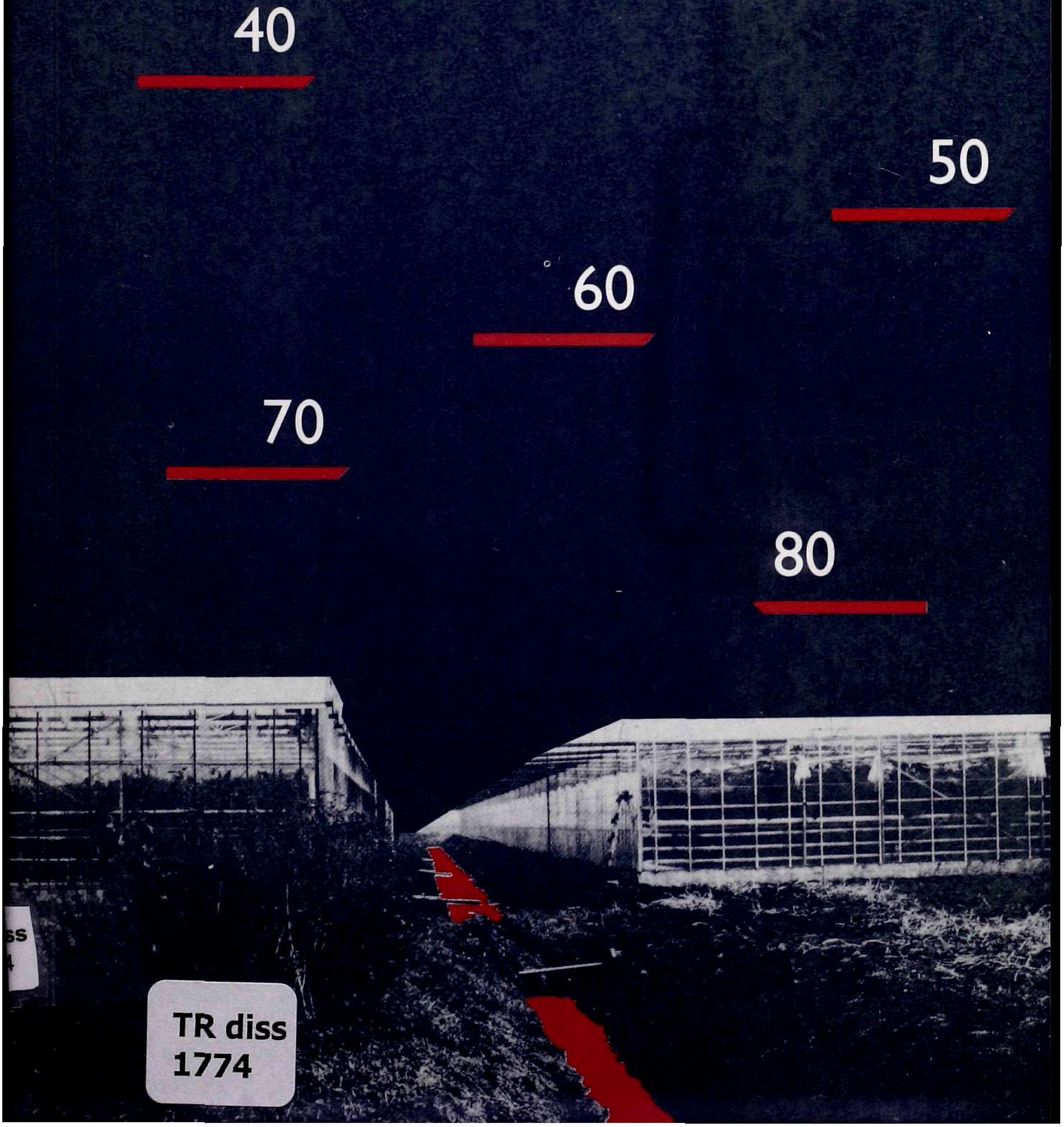
Some aspects of the design and operation of water management systems

C. VOLP



SOME ASPECTS OF THE DESIGN AND OPERATION OF WATER MANAGEMENT SYSTEMS

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Some aspects of the design and operation of water management systems

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ABSTRACT

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Some aspects of the design and operation of Water Management Systems. Delft University of Technology, Faculty of Civil Engineering, Sanitary Engineering and Water Management Division, Delft, the Netherlands. 173 pp., 37 eqs., 16 tables, 42 figs., 122 refs., Eng. and Dutch summaries.

This study deals with the design of water management systems. Attention is focussed on that phase in the design process in which variants are developed and evaluated with emphasis on the evaluation. The proposed evaluation procedure combines an economic evaluation with an evaluation of the hydraulic qualities. The economic evaluation implicates the determination of the internal rate of return but reduces to the determination of the net present value of the cost if no agricultural production level can be defined. The evaluation of the hydraulic qualities involves the steady and non-steady design criteria for various land uses or for various return periods. The evaluation is presented in a balance sheet where the economic evaluation can be balanced against the degree to which the design fulfils the design criteria. This enables the designer to develop various variants while evaluating. An example of the application of the procedure is given. In search of a method to determine a nominal agricultural production level which is related to the water management scenario, a model was developed where all aspects related to crop yield were included on the condition that nutrient supply is optimal. All aspects include workability, crop development, harvest conditions etc. This model makes it possible to quantify the effect of different water management scenarios on crop yield. Two examples are presented where the model is applied.

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DESIGN

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PREFACE

This thesis results from the CT-12.82.18.01 Research Project of the Delft University of Technology. The research work was carried out at the Sanitary Engineering and Water Management Division of the Faculty of Civil Engineering, the Netherlands. Although the thesis is not a composition of a set of papers, some of the research work has been published before. It concerns especially the subject of Section 3.2.

I would like to thank all those who supported and assisted me during this study.

In the first place I wish to express my gratitude to professor Wil Segeren for taking the initiative to create a place at the Technical University where this research work could be done. The freedom he gave me in the course of the study was greatly appreciated. In addition I am indepted to him for the valuable discussions we had and his willingness to act as promotor.

Together with Wil Segeren I would like to thank Jan Luijendijk and Piet van der Kloet who contributed much to my work, especially at the definition stage where they helped me with their guidance and stimulating discussions. Futhermore I would like to thank Bart Schultz, Bert Smedema, Koos Neefjes and Harm Albert Zanting for their valuable comments on the contents of this thesis. Special thanks goes to Reinder Feddes whose support at the last stage of the project was determining for finalizing this thesis. His willingness to help at the beginning of the project and his comments on one of the last concepts have been of great value and evidently contributed to the improvement of the final thesis.

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I am most grateful to Peter Blumenthal for turning my 'English' of one of the final concepts into Enlish that is generally recognized and understood as such. This was confirmed by Edward McKyes who found time to read the final concept and who came up with only a few errors. His correction work is appreciatively recognized. Great appreciation goes to Frances Clark for the lots of typing work that she has done with great patience. The same holds for Ruud Nicolaas for his efforts in making the drawings in rather a short period of time. Furthermore I want to thank Brigitte Beenen for the nice design she made for the cover of this thesis. Leni Trouw must be mentioned here for the screening work she did on the list of references.

The hospitality of the people of the Lioba Monastry is generally acknowlegded. They provided me always a warm and quiet chamber in the right atmosphere.

CURRICULUM VITAE

The author was born in Amersfoort, the Netherlands on 14 January 1953. He studied at the Delft University of Technology, Faculty of Civil Engineering, from 1971 to 1981. He graduated in civil engineering and hydrology with the master of science theses 'The significance of rainfall runoff relations' as major and 'Calibrations of various weirs' as minor subject. After his military service with the Royal Navy he was employed in the Sanitary Engineering and Water Management Division of the Faculty of Civil Engineering at Delft University of Technology from 1982 to 1986, where he developed the models discussed in this thesis. In addition to his research work he gave guidance to students in preparing their master of science theses and lectured. Since 1986 he has been on the staff of the executive bureau of SAMWAT - the cooperative association of universities, institutes, governmental, provincial and regional departments and consultants in the field of water management - the aims of which are the coordination of the participants' research activities and results and the dissemination of information.

To Marianne,
Anne and Nicolette

CONTENTS

CHAPTER 1 SCOPE AND ESSENCE OF THE STUDY

1.1 Introduction

This study deals with the design of the main components of a water management system. Though some results may conceivably have a wider application, the study is essentially based on the polder type water management system of the Netherlands. The main components of a water management system are understood to comprise the primary, secondary and, in a limited number of cases, tertiary system canals, pumping stations, weirs and culverts. The restriction to the polder type water management system is made here especially because of the underlying assumptions of the approach followed for the determination of the relation between water management and agricultural production. In polder type water management systems the assumption can be made that the level of the open water is (almost) horizontal and that the hydraulic head differences needed to transport the water are negligible. In that case a reservoir approach can be adopted for the determination of the open water level.

In the design practice in the field of water management the use of the computer as an aid is widespread. The computer is used to solve a great variety of problems ranging from groundwater flow to water allocation problems and from multi-reservoir operation problems to crop growth and rainfall/runoff simulation. Though the computer is almost indispensable, it is not yet used for computer aided design in this field as it is used in other fields like electrical, mechanical, marine and aviation engineering. This may be caused by the fact that in contrast to other fields of engineering the duration of the design process is long, and generally measured in years rather than months or even weeks. The design of water management systems passes through several stages. The number of stages that can be distinguished depends on the scale of the design process. A well known breakdown of the process resulting in a detailed design of a water management system, distinguishes three stages (Novak, 1987):

. a reconnaissance stage during which a pre - feasibility study is made, the main objective of which is to identify the feasibility of the proposed project on technical and economic grounds. Studies at this level are mainly based on existing information but may also include some field work.

. a semi-detailed stage during which a feasibility study is made, various plans from the reconnaissance study are worked out into a preliminary plan to be presented to decision makers for the final choice. Studies at this level need more detailed data.

. the detailed stage during which the final detailed design is made. This final design includes a list of quantities and the preparation of tender documents.

The three items mentioned do not describe the design process in detail but merely describe the stages the designer has to pass to obtain the necessary support for the project proposed. Support means here the authoritative support of the whole society where aspects of planning, ecology and economy are included. The duration of the considered design process can scarcely be shortened by applying faster design tools because the total duration is determined by the time necessary to obtain approval of and allocation of funds for the project by the organizations, ministries or departments involved. During the second and third stages, though, a period can be distinguished during which some time can be gained by applying computer programs specially developed for this *purpose: the sub-stage of the development of various variants of the* project for the final evaluation.

It is essential in this sub-stage that this part of the design process is a cyclic process where various variants are developed and judged by their merits in order to enable a choice between acceptance as a realistic variant, rejection, and/or acceptance as a base for another variant. Various kinds of computer programs are used in this stage to determine the performance of components of the system, and to a lesser degree for the evaluation of the performance of the total project design which is still often carried out manually. Especially in this sub-stage the use the computer bears several advantages. The often heard advantage of a saving of time can be questioned since the development and/or the modification of computer programmes and the data colllection requires often more time than expected. More important is, that the proposed variants would possibly gain in quality, since more variants could be taken into the consideration in a shorter time. In this cyclic process the judgement

of the variants by their merits, being the evaluation stage in this sub-stage, plays a key role.

1.2 The evaluation

The evaluation of the performance or the qualities of a variant should be carried out after the main goal(s) for the design has (have) been formulated. In the field of water management it is often relevant to formulate as a main objective for the design that inundation should be avoided. In addition it is often specified that under certain conditions a temporary inundation at a specific time of the year is not unacceptable.

The translation of this objective into a set of design criteria may take the form of:

. Water levels that must not be exceeded, considering a situation with a given return period. The situation is defined by data covering the time of the year, water levels, precipitation and rainfall runoff relations.

. Design runoff discharges based upon long experience that are presumed to have a certain return period and have to be controlled within water level limits.

The water level limits that play an important role in the above design criteria are often based upon traditional and/or empirical grounds. The design criteria formulated above demonstrate the difficulties of their formulation. The design criteria should be formulated in relation to the objectives aimed at water levels and velocities. For this purpose it is of the utmost importance that the objectives are defined as clearly as possible. This can be achieved by formulating an objective as:

. to manage the water levels and velocities in all points of the system in such a way that the benefits of the areas served are maximized with respect to one or more user groups .

If there is no positive relation between water levels and benefits for a certain kind of land use the objective can be formulated alternatively as follows:

. to manage the water levels and velocities in all points of the system in such a way that the damages to the *interests of the user groups of* the areas served are minimized.

In addition to this first main objective for the design the second main objective concerns the economical consequences of the design:

. the benefits are to be maximized on condition that the costs of the system are to be minimized.

In view of the above defined objectives the evaluation comprises:

. the expected benefits;
. the estimated costs.

The expected benefits

The definition of the expected benefits within the framework of this thesis, depends mainly on the kind of land use practiced on the areas served. For agricultural areas, benefits are the yields of agricultural products, varying from potatoes and grains to beef, milk and wool. These agricultural benefits depend largely on the water level and its variation during the year. The relationship between water levels and benefits varies during the year and also depends, among other things, on the soil type, the climate, the drainage conditions, farm management and type of crop. Much research work, especially in the past decades, has been focussed on the determination of this relation. One approach in determining this relation uses a computer model to construct, by simulations under various conditions, the relationship between the agricultural benefits and the type of water management. This subject is treated extensively in Chapter 2. For land use other than agricultural the establishment of this relationship is hampered by a lack of knowledge and/or a lack of measured data. In other cases this relation depends strongly on the situation under consideration. In urban areas the economic consequences of the occurrence of high water levels range from the capitalized loss of working hours due to inaccessibility of roads and subways on the one hand to the loss of furniture, carpets and stocks and to damage affecting road structures, subways and buildings on the other hand. In literature only some English and American information can be found, on damage due to inundation above ground-floor level in buildings with functions that range from housing to offices and industry. There is still a lack of knowledge concerning the range from the normal drainage base to ground-floor level. In urban areas the design objectives are determined by the many functions of the watercourses in urban areas. Apart from the transport and storage function, the watercourses in urban areas may have an aesthetical function, being an important element for the urban planner. This function can be of predominant importance in determining the dimensions of the watercourses. Maximum permissible water levels are determined by the levels of the bankslope, roads, parking lots, cellars and basements and finally the level of ground-floors of houses, offices and factories. For nature reserve areas, which often include canal banks and verges, many relationships are still unknown. The same holds true for recreational areas. A complicating factor is that these two kinds of land use are sometimes found in combination. Furthermore, in such areas the water level rise is often not important. The water level, with respect to ground level, is to be kept high. More serious damage can be expected of a water level fall than of a temporary inundation. For other areas such as glasshouse areas the relationship with the water level is either important or absent. Glasshouses in the Western part of the Netherlands for example often have a pumped sub-surface drainage system of their own. In these cases the groundwater level in the glasshouse is not, or not appreciably influenced by the water level in the watercourses. Not all glasshouses have this provision, and in those cases the groundwater level is indeed influenced by the water level. In the former case the relationship between the type of water management and the benefits is fairly simple up to the level of inundation of the glasshouses. In the latter case the relationship depends, among other things, on the drainage intensity, the sensitivity of the crop, the rooting depth and the climate in the glasshouse. This applies on condition that the crop is grown on the ground and not on tables, on wood-wool cement slabs or other media, in these circumstances the relationship is similar to the relationships of the former case.

Besides the uncertainties in the determination of the relationship between water management and benefits for each type of land use there is the aspect of combined and possibly conflicting interests of the managers involved in the same location or locations adjacent to each other, which increases the degree of complexity for the designer of an evaluation tool. In those cases the one party will demonstrate that much damage will result from a particular measure while the opposing party will describe that the same measure will have no effects on the basic situation on that particular location.

The above given aspects compel the designer of an evaluation tool to choose between the following two possibilities:

- To construct with the best possible means and insights a relation for those land uses. The relative importance of the relationships or parts of the relationships can be determined by means of a sensitivity analysis for each of the relationships constructed. The results of this analysis must be taken into account in the evaluation of the final set of variants. There is a danger that this method will end up with a large number of relationships, especially if all the factors under study are considered in their own dimensions. This will not simplify the evaluation of a variant, let alone the final evaluation of the final set of variants.
- To formulate, or allow land users themselves to formulate, criteria in terms of water levels that are not to be exceeded or may not be exceeded longer than a given period of time, this at a fixed return period. This solution has the benefit of simplicity combined with the fact that it does not deviate from the usual design practice. But, more important, it does offer the possibility that this relatively small set of criteria as well as its significance to the final set of variants, can be easily evaluated.

In this thesis the second option has been chosen for these types of land use. In recapitulation the above reasoning leads to the following conclusion: an evaluation tool in a programme for the design of water management systems should demonstrate the benefit side in tons per unit area, possibly even translated into monetary units such as guilders per hectare, for the agricultural sub areas of the total area. For other land uses in the project area, the demonstration of the benefit side should preferably take the form of a score list presenting the degree of fulfilment to an established, but not necessary static, set of design criteria. This subject will be treated in more detail in Chapter 4.

The estimated costs

The estimated costs for the construction of a system of watercourses, pumping station, culverts etcetera for the purpose of an economic evaluation should include the investment costs, the maintenance costs and the exploitation costs. If the benefits can not be expressed in monetary units, the economic evaluation is reduced to the presentation of the net present value of the costs for a given time horizon and rate of discount. If benefits can be expressed in monetary units, as is the case for agricultural benefits, the economic evaluation may result in the internal rate of return of the project under study.

The combined approach

Preferably both aspects of a variant, the benefits and the costs, should be presented in combination, to enable the designer to relate the two aspects. Based on the above reasoning, in this thesis the presentation of the benefits and the costs is translated into, respectively, the presentation of hydraulic qualities, and the net present value of the costs or the internal rate of return. This combined presentation offers the designer the possibility to develop applicable variants which fulfil all criteria, relate them to the corresponding costs and choose the best (the least expensive) variant. But this presentation offers more, which in the end will turn out to be the most important feature. It will be relatively simple to perform an extensive analysis for the sensitivity of a variant to varying rainfall/runoff relations and varying volumes and intensities of rainfall, given a certain set of criteria. This can be an important feature in those cases where rainfall runoff relations and design conditions are (partially) unknown and design criteria are based on traditional grounds. The second important application is to confront the manager, proprietor or administrator of a given land use with the financial consequences of his criteria. Or, better still, to confront him with the financial consequences if his criteria are not met to the full 100% in all places.

1.3 Purpose of the study

Considering the reasoning of the preceding sections the aim of the study can be formulated as follows:

> **To develop a tool for the designer, the manager or administrator of water management systems which makes it possible and relatively easy to develop at the computer terminal various variants for the project under consideration and evaluate the hydraulic qualities of that variant in combination with the economic consequences.**

The benefits of such a tool are basically due to the fact that more variants for a project under consideration can be developed and with a good presentation of the different effects of measures in the design, more applicable variants for the final evaluation will possibly lead to an ultimate design of higher quality.

The fast working screening evaluation method as defined above concentrates on the investigation of the performance of the system and the economic consequences. The performance refers here to a full description of the water movement in such a system. The economic evaluation should give the costs of any relevant variant. The benefits may possibly, but not necessarily, be incorporated in this economic evaluation.

This tool is pointed at the dimensions of the watercourses, at the degree to which these watercourses with that dimensions can comply with the water quantities under design conditions, at the costs to make and operate the system and the benefits as far as the agricultural production is concerned. A programme that enables the above aspects to be studied can be composed of the following modules:

. a module for the description of non-steady flow in a system of watercourses. *Non-steady flow is chosen because this feature offers the* possibilities to account for phenomena such as dynamic storage in the system;

. a module simulating the runoff of various kinds of surfaces;

. a module by which the water levels which results from a simulation are confronted with the design criteria;

. a module for the determination of the costs of all parts of the system as well as of the whole system, and possibly, if benefits are known or can be estimated, the internal rate of return;

. a module for the determination of the agricultural benefits in relation to the operation rules applied including groundwater flow and the relation between the groundwater and the water in the watercourses.

The above enumeration should be considered with care because if applied as the requirements for a model to be developed it may imply unnecessary complex requirements. The reason for this is one of computer time efficiency and of model characteristics.

For the determination of the computer time necessary to describe the various processes, two aspects must be taken into account. The first aspect is that the time step of the calculations is determined by the process demanding the shortest time step. In this case roughly two processes can be distinguished:

. The process of the water movement through a system of watercourses. This process can best be simulated if a timestep is chosen in the order of minutes.

. The process of crop growth and the water movement through the soil. This process can very well be simulated using a timestep in the order *of hours or days.*

The second aspect relates to the fact that for the simulation of crop growth total computation time must cover at least the growing season but preferably the whole year. Events of sufficient importance for simulation with the hydro-dynamic model will usually not last longer than one, two or three days.

The above considerations mean that a model including both processes should simulate events lasting at least months, and would need a calculation time step of minutes. Such a model would be very time consuming and consequently expensive. On top of that, the fact has to be taken into account that models of water management systems generally include more than one node where water levels as well as crop growth must be simulated. The more nodes, the more crop growth models would be necessary. Besides the already mentioned calculation time, the disadvantages are related to the fact that such a model requires much more data for not only data *concerning the system but also concerning the agricultural production* would be incorporated. The result would be a less flexible and less distinct model due to the greater number of parameters.

The solution to this problem depends on the kind of problems that is aimed at. Basically it depends on whether a full non steady description of the water movement through the system of watercourses is necessary.

If the calculation of the water movement can be accomplished with a time step of hours a possible solution might be to run one model where two different time steps are used, one for the water movement through the watercourses and one for the water movement through the soil profile.

Developments of recent years which follow this reasoning are the models PREDIS (Crebas, 1984) and SIMPRO (Querner, 1987). These models combine a non-steady description of flow through the system of watercourses with the non-steady description of flow through the soil profile. The advantage of such an approach is that it is comprehensive, which may give a better insight into the consequences of a particular set of water management regulations on a regional scale. The usefulness of the non steady description of the water movement through the watercourses may be questioned in these cases and often a steady state approach will give similar results.

In problems where the non steady character of the water movement through the watercourses is well pronounced, resulting in time steps of the calculation in the order of minutes, a solution to the dilemma is to develop two models. One of these would possess the qualities described above for the evaluating tool, the other would be dedicated to the determination of the relationship between water management and crop growth. The latter would be suitable to investigate whether it is possible to determine the relationship in the first place but also whether it would be relevant to combine the two models. This solution has been adopted in this study.

For the benefit of this thesis two models have been developed and their qualities will be discussed. The core model, the evaluating tool, was named the EWAS model. EWAS, a model for the design and **E**valuation of **WA**ter management **S**ystems, offers the options as described above. The second model which is suitable to establish the relationship between the open water level and the benefits for at least one kind of land use. It is intended to be utilized in the evaluation stage of the EWAS model, and was named the PRODU model. PRODU, a model for the determination of the **PRODU**ction of agricultural areas in relationship with water management, was designed to relate various water management regulations with the productivity of agricultural areas. The results obtained using this model can be expressed as a productivity level with the dimension "weight per unit area" [$t.ha^{-1}$]. If possible, these results may finally be used in the financial evaluation of EWAS.

EWAS and PRODU are two different models, and although the results of PRODU may be used in EWAS in some applications, and the results attained with EWAS may initiate further research with PRODU, these steps may not all be necessary.

In order to be able to answer the question whether or not PRODU and EWAS should be linked the following analysis is given, directed at identification of the cases in which the models should be linked as opposed to the cases in which they had better not be linked. For the analysis the aim of the study as previously stated is considered. The key conception in this aim is the design of water management systems. This conception deserves further consideration.

The activities in the design process are directed at the determination of the required set of transport and storage capacities within the boundary conditions of the area. These boundary conditions are the topography of the area, e.g. the soils and the climate, and they can be seen as the unchangeable parameters in the design process. The requirements for transport and storage capacities are related to the dimensions of the infrastructure and to the system's rules of operation.

The design is directed at the determination of the essential dimensions of the hardware of the system such as the lengths, the widths and the locations. For the determination of the dimensions it is essential to know the criteria for the design, the boundary conditions like the target levels, the maximum (and minimum) levels as well as, for instance, the permissible level variation but also the capacities for drainage from and supply to the area under consideration. The determination of these criteria is part of the design process, although they are often fixed by external sources. The criteria with their underlying rules of operation, (the software), specify to what extent the hardware may be used. Their determination should be carried out with care, for the opportunities to reach optimum design are derived from these criteria. The word optimum is used here in relation to costs, for the criteria give insight into the extent to which the dynamic storage can be used, and into the effects in terms of smaller storage in the channel system, in other words: the potential savings in investment and maintenance costs. Also, the word optimum is used here in relation to the benefits, because the determination of the rules of operation may result in higher yields or in lower yield depressions.

In order to obtain a clear presentation of the kind of problems in design in which PRODU and EWAS should be linked and of the cases in which they should preferably be used separately, a scheme is presented, Scheme 1.1. For the benefit of the analysis, a breakdown of the design problems in practice is combined with a rough division of the type of areas which may be subject of study. For the analysis regarding the conditions prevailing in the Netherlands it is sufficient to discriminate between the flat polder areas that are found in the north and west regions and the sloping areas in the east and south, as far as fixed boundary conditions are concerned. The breakdown of the design problems in practice distinguishes three levels of interest: the design of water management policies, the preliminary design of the infrastructure of water courses, and the detailed design of the infrastructure of water courses. The following reflections apply to the scheme.

The design of water management policies.

The subject of interest here is to determine the optimal water management policy in a particular area, or the determination of the boundary conditions for the design like the target levels, the maximum (and minimum) levels and the permissible level variation, but also the capacities for drainage from and water supply to the area under consideration. As far as flat areas are concerned the PRODU model is the appropriate tool for this field of interest. The examples of Chapter 3 illustrate this. For sloping areas the water movement through the soil and the channel system is more complex. Depending on the project PRODU should be linked with other models. In an extreme situation PRODU should be combined with a three dimensional groundwater model to simulate correctly the fluid transport through the soil from higher to lower areas and with EWAS because of the non-steady character of the transport through the system of watercourses. In scheme 1.1. this is indicated by COM.

The preliminary design of the infrastructure of water courses.

The same fields of interest are concerned here that are mentioned in the preceding paragraph, with the addition that now cases are considered in which areas with different target levels are (or should preferably be) connected, in order to transport the water from an area with a water surplus to an area with a shortage of water. For flat areas it may be stated that PRODU is the appropriate tool as well, if it is upgraded with a module which directs and allocates these quantities of water. Therefore PRODU$^+$ is indicated in the scheme. For sloping areas a similar reasoning as given in the preceding paragraph leads to the indication of COM in the scheme.

design of:	flat area	sloping area
water management policy	PRODU	COM
preliminary design of infra-structure	PRODU$^+$	COM
detailed design of infra-structure	EWAS	EWAS

Scheme 1.1. Review of the kind of problems in design in which PRODU and EWAS should be linked as opposed to separate use. In this scheme the acronym PRODU is used if PRODU can be used separately, PRODU$^+$ is used if a combination of more PRODU models must be used, EWAS indicates the use of EWAS separately, and COM indicates the cases in which PRODU and EWAS should be combined.

The detailed design of the infrastructure of water courses.

Optimal dimensions as well as the design of the infrastructure of channels, weirs, pumps, etc, are the objectives here. For this kind of problems EWAS is the suitable instrument. For flat areas the question of whether or not a steady state approach might be sufficient depends on the rainfall runoff relationship components in the area. In the scheme EWAS is marked because of the test *and the economic module*; it applies in cases in which the non-steady approach is needed. The same holds for sloping areas.

Recapitulating it can be stated that the models PRODU and EWAS can be applied especially in flat areas and do not have to be combined for such applications. In addition to this it may be stated that the approach followed in these models should preferably be adopted as well in models suitable to be applied in sloping areas.

1.4 Structure of the report

The scope of the study, set forth in the previous sections, may be considered to be the thread of this report.

In the following chapters the PRODU model is described first, in Chapters 2 and 3, to investigate the relationship between water management and agricultural production. Secondly the evaluating tool, the EWAS model, is described in Chapters 4 and 5.

The two chapters for each model are similar in contents. The first chapter describes the backgrounds of the model, the second chapter is dedicated to applications of the model. Both chapters 2 and 4 are similar in content. After an introduction where the model in question is placed in a wider context, a model description is given. All aspects of the model are dealt with against a background of existing theories and practices. After the description of the application possibilities of the models in chapters 4 and 5, the final section of these chapters is dedicated to a discussion of the strong and weak points of both models.

In Chapter 3 two examples are given of the research that has been carried out with the PRODU model. These examples have been chosen in such a way that they illustrate the usefulness of the application of the PRODU model for questions which arise in practical situations. The first example discusses the different aspects of the choice of the date for switching from winter level to summer level. The relation of the open water percentage to potato yield is being studied in the second example.

In Chapter 5 an application is described where EWAS is used as a designing tool. It treats the case of the Erica glasshouse area, a case where the non steady character of the water movement through the watercourses under design conditions plays an important role.

In the sixth chapter the relevance of the work of this study for practical purposes is discussed and a glimpse is given on future developments.

CHAPTER 2 WATER MANAGEMENT AND AGRICULTURAL PRODUCTION

2.1 Introduction

In order to quantify the effects of different kinds of measures in the field of water management with respect to agricultural production, a computer model can be the appropriate aid to establish those particular relationships based upon series of simulations. An important quality of such a model is that the agricultural production is expressed in a quantity per unit area for comparison purposes. Furthermore the conception of water management should include the total of the various design parameters in this field, i.e. discharge capacity (pumps, weirs), storage capacities (surface water area, reservoirs), crest levels, "on" and "off" statuses for pumps, as well as water supply capacities. Also operational matters should be included such as the date to switch from winter to summer level (or vice versa) and the date to start or stop water supply. What is desired is an agricultural production model where water management has been incorporated.

In mid 1985 many agricultural production models were operational. In a comprehensive review by Slabbers et al. (1979), 111 references focussing on the water crop yield relation are mentioned where 12 references indicate the development of an agricultural production model. Since that date more models have been developed and a literature search on agricultural production, crop growth simulation models dated June 1987 resulted in a list of more than 45 articles describing water crop yield models registered after 1980. These models are dedicated to determine the water crop growth relation for a wide range of crops varying from grasslands, tulips, sugarcane, cucumbers, alfalfa, cotton, wheat and potatoes to pine and elm-ash-cottonwood forests. The models have been developed and used for general purposes such as irrigation scheduling programme optimization, farm management optimization and regional water allocation studies, as well as for research purposes where growth is simulated under extreme conditions as low temperatures, arid climate conditions and saline irrigation-water supply.

In the Netherlands a number of hydrological models have been developed by research institutes, government services and universities. The development of these models was speeded up after 1976. After this dry year, some regional and nationwide studies were initiated (Rand Corporation, 1983; Werkgroep Watervoorziening Drenthe, 1979; Commissie Bestudering Waterhuishouding Gelderland, 1980), later followed by regional studies for water supply (Boheemen, 1981; Herinrichtingscommissie Oost-Groningen en de Gronings-Drenthse Veenkolonien, 1983; Boheemen, et al., 1983) and studies of the consequences of groundwater extraction, for which reference is made to the report of the LAGO working group giving a review of the agricultural aspects of groundwater extraction (Commissie Grondwaterwet Waterleidingbedrijven, 1984). The most elaborate water allocation model was developed in the studies for the Policy Analysis of Water Management for the Netherlands (PAWN) in which the available water is assigned on a nationwide scale to the various regions and users. In this study the utilization purposes include shipping, salt wedges, power plants, industry, agriculture and drinking water supply (Rand Corporation, 1982). On a somewhat smaller scale, but taking into account the same user groups as in the PAWN study, a package of models was developed by the Provincial Public Works Department of Gelderland (CHO-TNO, 1981). Considering the models which evaluate the gains or losses due to quantities of water supplied to agricultural users, there are the submodels DISTAG (District Hydrological and Agricultural model) and UNSAT (later replaced by the model MUST) in respectively PAWN and GELGAM. Both models operate with time steps of a 10-day period especially suitable to evaluate drought damage. Within the framework of the land consolidation works of the Ministry of Agriculture, agricultural models were developed by the Institute of Land and Water Management Research (ICW). The agricultural models, a grassland and an arable land model, were developed in connection with the HELP evaluation method. The objectives for the agricultural models of the Staring Centre (former ICW) are to relate groundwater levels and water management measures to agricultural production (Feddes and Wijk, 1976; Walsum and Bakel, 1983; Feddes et al., 1984; Feddes, 1985; Bakel, 1984, 1986).

Whereas the agricultural models of the PAWN study and the Provincial Public Works Department of Gelderland are especially suitable for determining the consequences, in terms of yield loss, of moisture deficiency, the objectives of the agricultural models of the Staring Centre (SC) are more comprehensive. This appears from Fig. 2.1 showing

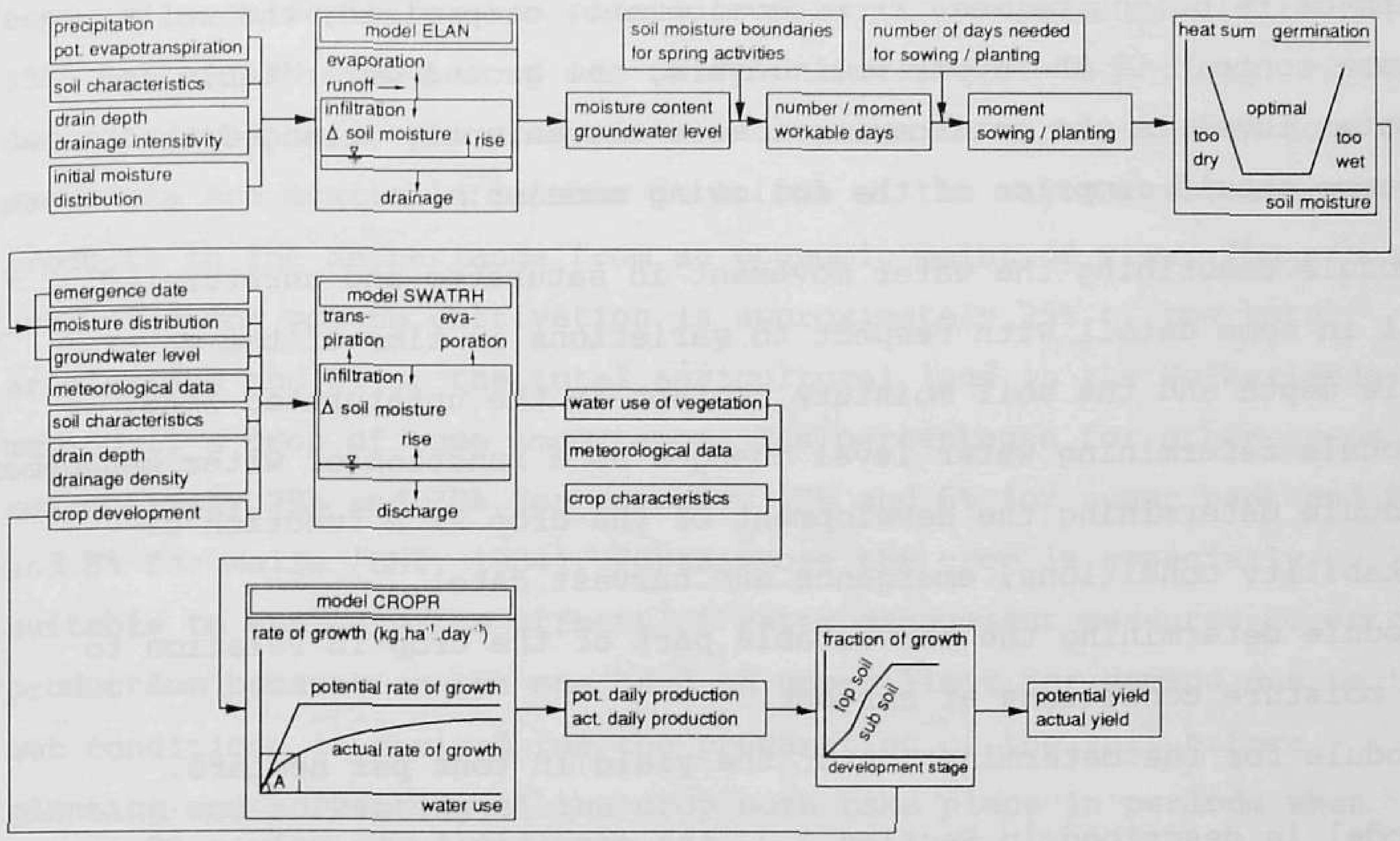


Fig. 2.1 The package of models for determining the relation between water management and agricultural production (Wijk and Feddes, 1982)

the package of models for determining the productivity of arable land. The diagram illustrates the wider scope of the research. The SWATRE model (Soil Water and Transpiration Extended) in this package can be compared with the MUST and DEMGEN models of the Provincial Public Works Department of Gelderland and the PAWN study respectively, with respect to their simulation of unsaturated groundwater flow and transpiration processes. This comparison was carried out in 1984 together with two similar models, SOMOF and UNZAT, of the Delft Hydraulics Laboratory and the National Institute for Drinking Water supply, respectively. For the description of the models reference is made for UNZAT to Drecht (1983), for SOMOF to Gilding (1983), for DEMGEN to Abrahamse et al. (1982), for MUST to Laat (1985), and for SWATRE to Belmans et al. (1983). The results of the model comparison were reported at the fourth seminar of the Committee for Hydrological Research TNO in October 1984 (Hooghart, 1985). In addition to these five models the models of Schultz (1982), Minderhoud (1982), and Kaland (1984) should also be mentioned. In the light of the above it was decided not to develop a new model but merely adopt components of existing models possibly with some adjustments, to arrive at a new model which after some extensions could fulfil the requirements of the type of programme described at the beginning of this introduction. Such a programme should describe the complex of relations between crop development, crop yield, the soil moisture content in the upper soil layers, the ground water table and the water level in the watercourses. More conveniently arranged the programme should comprise of the following modules:

. a module describing the water movement in saturated and unsaturated soil in some detail with respect to variations in time of the water table depth and the soil moisture content in the unsaturated zone.
. a module determining water level changes as a function of water management.
. a module determining the development of the crop as a function of workability conditions, emergence and harvest date.
. a module determining the harvestable part of the crop in relation to the moisture conditions at harvest.
. a module for the determination of the yield in tons per hectare.

The model is described in Section 2.2, the sensitivity to values of some parameters in Section 2.3. The method applied to evaluate the results is treated in Section 2.4. The applicability of the model is illustrated with some examples in Chapter 3. The value of the model is discussed in Section 3.4.

2.2 Model description

2.2.1 Introduction

This section deals with the PRODU model, a model for the determination of the **PRODU**ction of arable land in relation to water management in polder areas and areas having the same surface water level. PRODU is a programme which determines the total yield in tons/hectare and in relation to time: the course of the actual and potential evapotranspiration, the actual and potential dry matter production and the actual and potential potato production in relation to data defining a water management system, meteorological data, crop and soil data. In the calculations the determination of the soil workability conditions and the seedling emergence are incorporated. As will be clarified in the following sections the research of the Institute of Land and Water Management Research at Wageningen within the framework of the HELP studies has been followed in broad outline for the development of the PRODU model. The model is especially suitable for the potato crop as it is operational from May 1985 on the IBM computer of the Delft University of Technology. PRODU is dedicated to the potato crop since much research work has been done and many data are available because it is one of the major agricultural products in The Netherlands from an economic point of view. The annual area used for potato cultivation is approximately 25% of the total arable land and 8% of the total agricultural land in the Netherlands, making it a crop of some importance. The percentages for other crops are respectively 29% and 10% for cereals, 17% and 6% for sugar beet and 22% and 8% for maize (LEI, 1984). Furthermore the crop is especially suitable to evaluate the effects of water management measures to crop production because on the one hand an upper limit for damage due to too wet conditions is derived for the preparation of the soil before planting and harvesting of the crop both take place in periods when access to the land can be problematical under Dutch conditions if water management is not optimal. On the other hand an upper limit for damage arising from too dry conditions is considered because rooting depth is relatively small as compared with that of maize and wheat. In addition, the water uptake efficiency of the roots decreases earlier due to drying of the soil as compared with other crops.

The model can be made suitable for other arable crops with a certain amount of modification. To modify it for grass will require more effort, because for determining the total yearly production of grass the relation between production and use of the grassland and the damage to the crop caused by trampling of the cattle, have not yet been adequately solved.

To meet the requirements the PRODU model, schematically represented in Fig. 2.2., consists of seven modules which are enumerated below and treated in detail in the following sections.

1. A part describing the water movement in the unsaturated soil, the flux to the ditches, subsurface drainage and drainage to deep aquifers and the evapotranspiration flux. For this part, which forms the heart of the programme, SWATRE has been adopted (Belmans et al. 1983). The main argument for this adoption is that the non-linear partial differential equation is solved by a finite difference

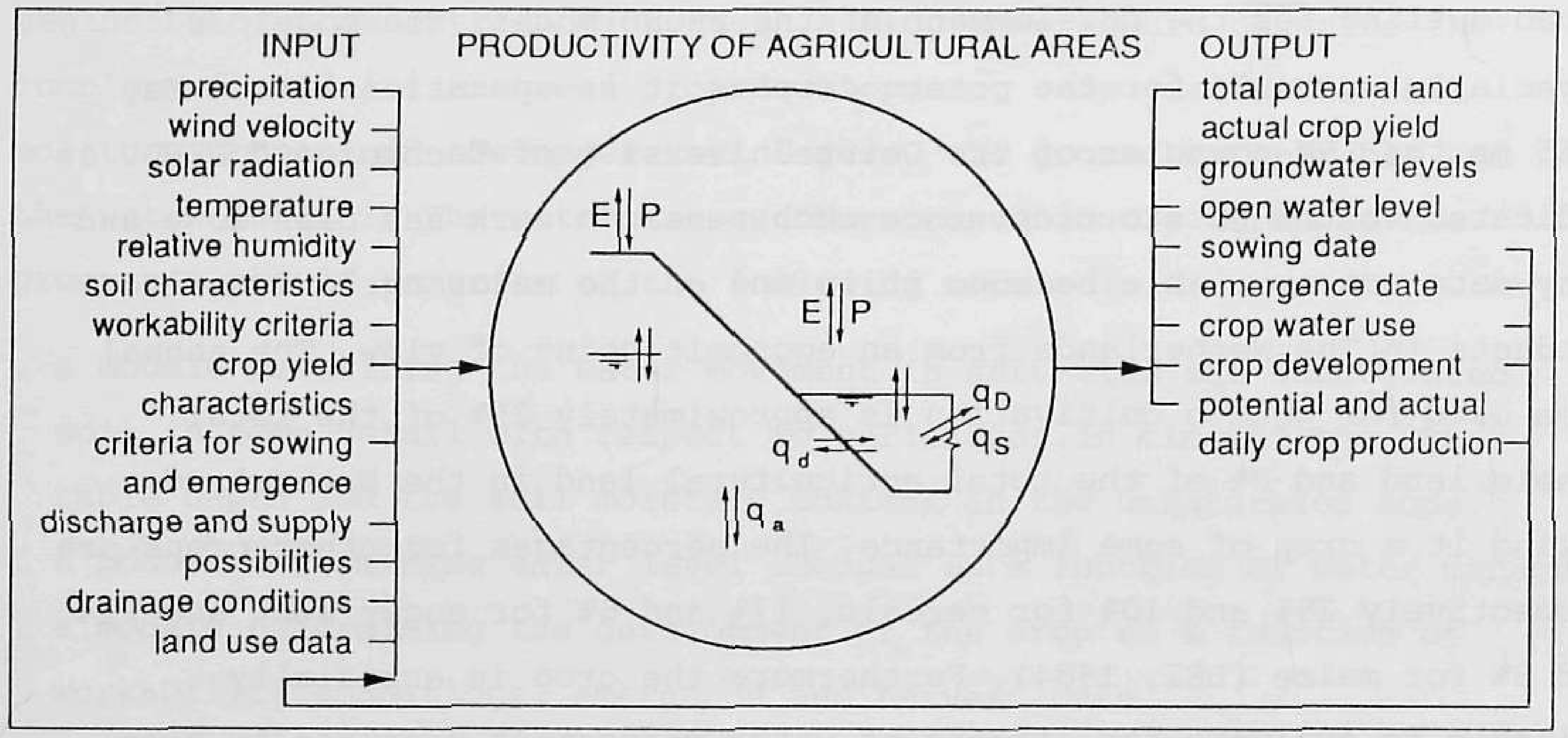


Fig. 2.2 Schematic representation of PRODU, a model for the determination of the **PRODU**ction of arable land in relation to water management. Where P is gross precipitation, E is evapotranspiration rate, q_d is flow to/from ditches/drains, q_a is flow to/from the deep aquifer, q_D is extracted quantity from the system, q_S is supplied quantity to the system.

procedure, enabling the detailed calculation of the moisture content with respect to depth as well as to time. A second argument is that the model has been tested and validated, with satisfactory results. SWATRE is briefly described in Section 2.2.2.

2. A module for determining the open water levels in the system of watercourses in relation to the water management scenario and the infrastructural characteristics of the area. The module is described in Section 2.2.3.
3. A module for the evaluation of the workability conditions. This module is discussed in Section 2.2.4.
4. A module determining the emergence date if the planting date is known. This module is dealt with in Section 2.2.5.
5. A module for the determination of the depth of the root zone, the crop height, the soil cover and the ratio of the harvested part of the plant to total production in relation to time. In Section 2.2.6. this subject is treated in detail.
6. A module for simulating crop losses due to too wet conditions at harvest which is treated in Section 2.2.7.
7. A module for the conversion of potential and actual transpiration into dry matter production and potato yield. The CROPR programme as described by Feddes et al. (1978) has been adopted and is briefly described in Section 2.2.8.

2.2.2 Soil water movement

Introduction

For a comprehensive treatise on the SWATRE model with underlying conceptions, reasoning, validation and sensitivity analysis reference is made to Feddes et al. (1978), Belmans et al. (1983), Walsum and Bakel (1983) and Bakel (1984, 1986); for a general treatise on numerical models for unsaturated groundwater flow and transpiration the reader is referred to Koopmans (1985). For a comparison of the SWATRE model with other similar models see Hooghart (1985). A brief description of the model SWATRE is given in this section for the sake of completeness.

Basic flow equation

The flow of water in a one dimensional multi-layered soil root system can be described by (Belmans et al., 1983):

$$\frac{\delta h_p}{\delta t} = \frac{1}{C(h_p)} \cdot \frac{\delta}{\delta z} \left[K(h_p)\left(\frac{\delta h_p}{\delta z} + 1\right)\right] - \frac{S(h_p)}{C(h_p)} \qquad (2.1)$$

where: h_p = soil water pressure head [cm]
t = time [d]
C = differential moisture capacity [cm^{-1}]
z = vertical coordinate with origin at surface, positive direction upward [cm]
K = hydraulic conductivity [$\mathrm{cm.\ d}^{-1}$]
S = sink term, water uptake by roots [d^{-1}]

The sink term is described by:

$$S(h) = \alpha_S(h_p).S_{max} \qquad (2.2)$$

where $\alpha_S(h_p)$ is a prescribed function of soil water pressure head, see Fig. 2.3., and S_{max} is the maximum possible root extraction rate, defined as:

$$S_{max} = \frac{E_{tp}}{L_r} \qquad (2.3)$$

where E_{tp} is the potential transpiration rate [$cm.d^{-1}$] and L_r is the depth of the root zone [cm].

Equation 2.1. is solved in SWATRE with an implicit finite difference procedure. For that purpose the soil system is divided into compartments of equal height. The finite difference version of equation 2.1 has to be solved for all n compartments in the unsaturated zone. The constraints for the top and bottom compartments leaves n-2 equations with n-2 unknowns. In the application of the Thomas algorithm the time step is variable and determined by a given maximum change of moisture content, a given maximum time step or the condition that it must not be larger than 1.20 times the preceding time step. It will be smaller or equal to the minimum of the three.

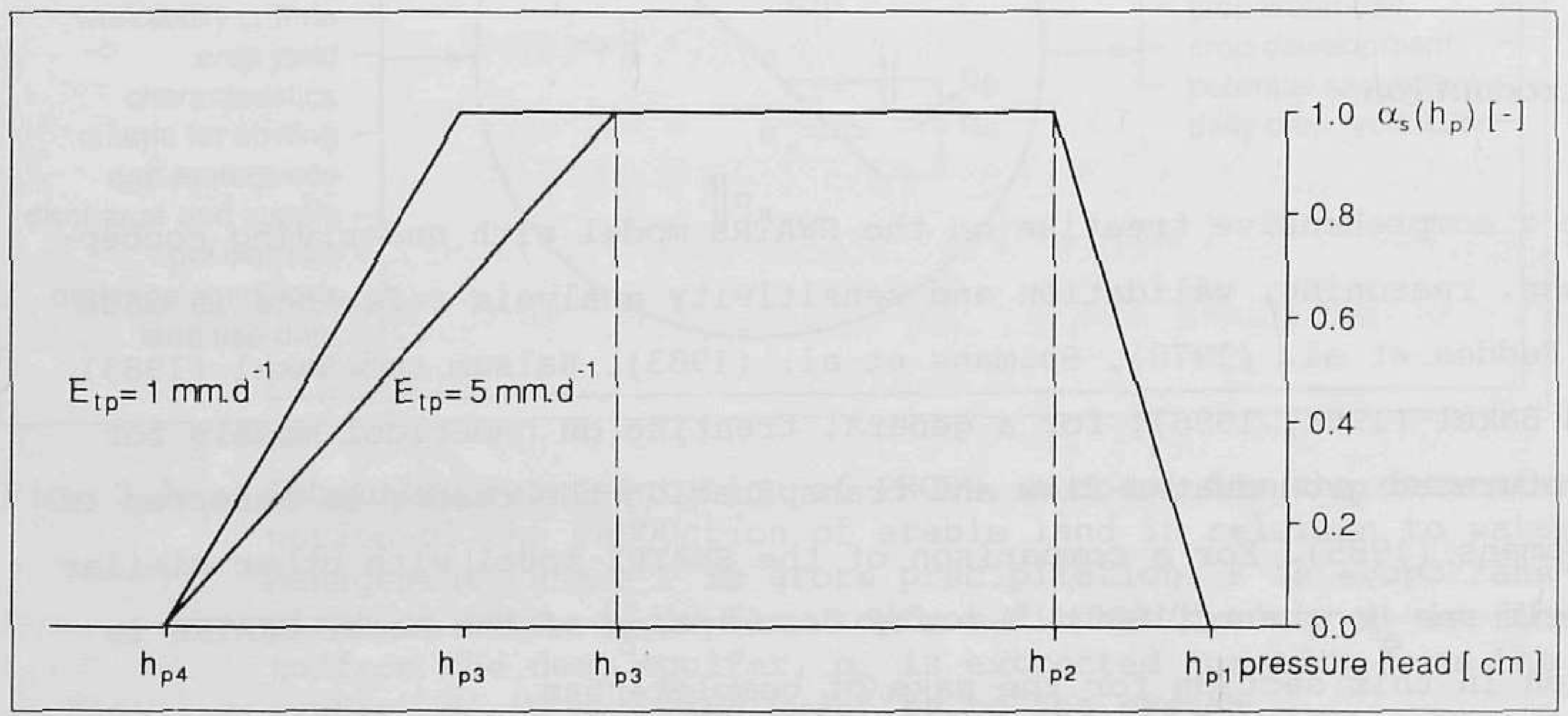


Fig. 2.3. General shape of the dimensionless sink term variable α as a function of the soil water pressure head h, based on Feddes et al. (1978). Where h_{p1} (anaerobiosis point), h_{p2} and h_{p3} (reduction point), h_{p4} (wilting point), for potatoes respectivily 45, 460, 16000 cm pressure head.

Boundary conditions at the top of the system

At the top of the system a maximum possible flux through the canopy E_{tp} is to be defined as well as a maximum possible flux through the soil surface q_{sp}. The value of E_{tp} can be computed as:

$$E_{tp} = E_p - E_{sp} \tag{2.4}$$

where: E_p = potential evapotranspiration [$mm.d^{-1}$]
E_{sp} = potential soil evaporation [$mm.d^{-1}$]
E_{tp} = potential transpiration. [$mm.d^{-1}$]

Three optional ways are available in the standard version of SWATRE version 1982 to calculate the quantity of E_p. In this study the modified Penman equation is used (Penman, 1948; Monteith, 1965; Rijtema, 1965; Feddes et al., 1978). The quantity E_{sp} is calculated as:

$$E_{sp} = \exp(-0.6 \, . \, I_L).E_p \tag{2.5}$$

where I_L is the Leaf Area Index, being a function of the soil cover which has to be known. This will be treated in Section 2.2.6. The potential flux through the soil surface q_{sp} is equal to the difference between the (reduced) potential soil evaporation and the precipitation reduced by the interception. The actual flux through the soil surface is governed by the actual transmitting properties of the soil.

Boundary conditions at the bottom of the system

The flux at the bottom of the system can be given directly as an input or can be calculated. For PRODU, the possibility of the use of the calculated flux determined by subsurface flow to drains, to ditches and to deep aquifers is the most interesting option. It offers the possibility to connect the model to an open water level controlled by pumps, wears or sluices. The bottom flux q_m is calculated in the programme as a result of in/outflow from/to ditches or sub-surface drains and downward/upward flow to/from deep aquifers (Fig. 2.4).

Flow to/from the ditches/drains is calculated as:

$$q_d = -(h_o - h_{f,m})/T \quad [cm.d^{-1}] \tag{2.6}$$

where: h_o = level of open water in ditch [cm]
$h_{f,m}$ = groundwater level midway between the ditches [cm]
T = drainage resistance [d]

According to Ernst (1962):

$$T = L_d w + L_d^2/8KD \tag{2.7}$$

where: L_d = spacing of the ditches [m]
w = radial resistance [$d.m^{-1}$]
K = hydraulic conductivity [$m.d^{-1}$]
D = average thickness of the aquifer [m]

For homogeneous soils and small variations in height of the phreatic surface the radial resistance w can be found from Ernst (1962):

$$w = (1/\pi K).\ \ln(D_0/B_w) \qquad [d.m^{-1}] \qquad (2.8)$$

where:
D_0 = thickness of the aquifer below the water level in the ditch [m]
B_w = wet perimeter [m]

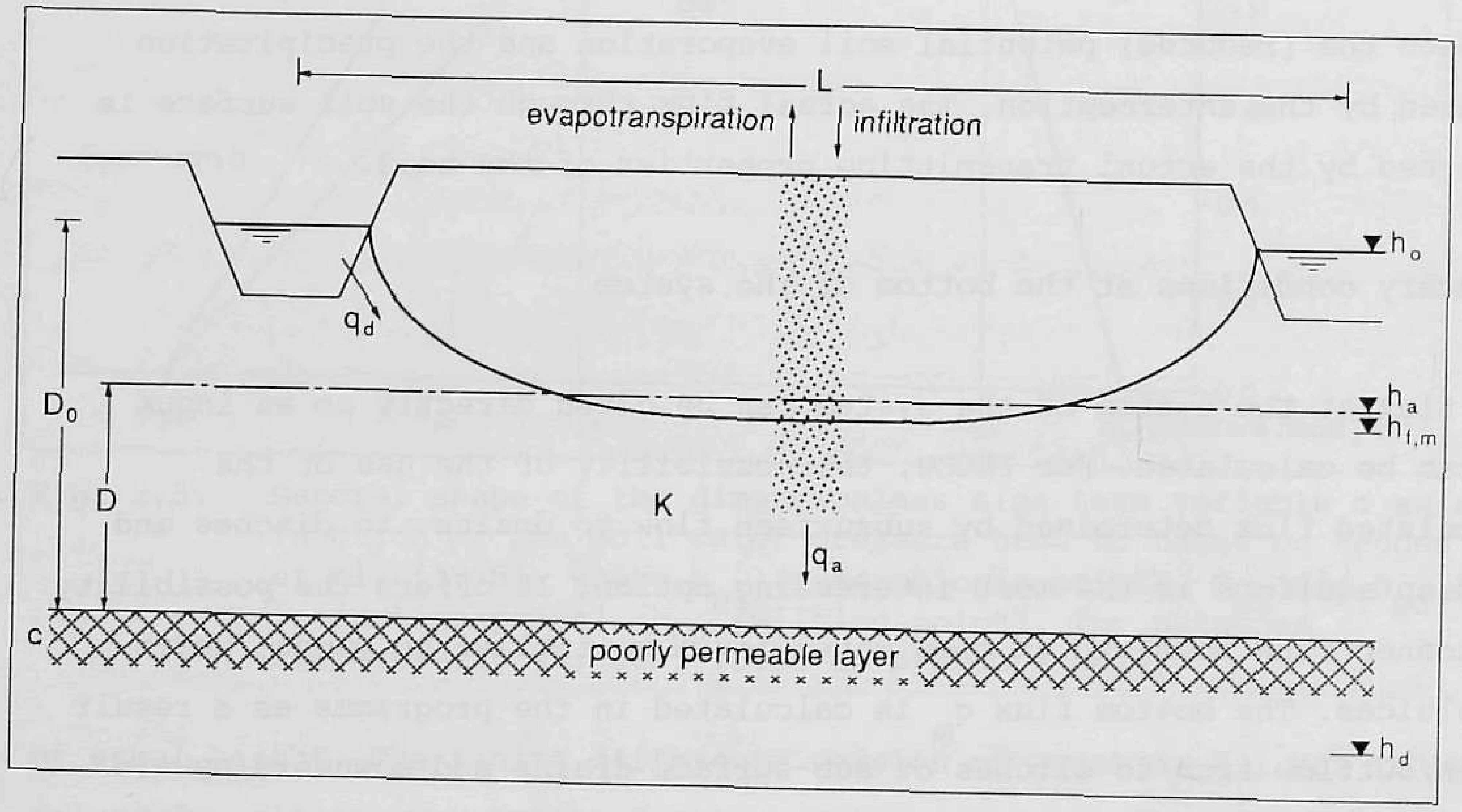


Fig. 2.4. Schematic representation of the flow situation in the case of sub-irrigation from the ditches and downward flow to the deep aquifers (Belmans et al. 1983).

The flow to the deep aquifer (seepage) is computed from:

$$q_a = -\ (h_a - h_d)/c \qquad [cm.d^{-1}] \qquad (2.9)$$

where:
h_a = level of the phreatic surface averaged over the area [cm]
h_d = piezometric level of the deep groundwater [cm]
c = vertical resistance of the poorly permeable layer [d].

The value of h_a can be obtained from:

$$h_a = h_o + \eta_i.(h_{f,m} - h_o) \qquad (2.10)$$

where η_i is a reduction coefficient depending on the shape of the phreatic surface. If this shape is parabolic, then η_i = 2/3. The vertical flux per unit area midway between the ditches, q_m, is now taken to pass the bottom of the soil profile system and is calculated as:

$$q_m = q_d + q_a \qquad (2.11)$$

2.2.3 Water management

The description of the SWATRE model in Section 2.2.2 shows how the moisture content in the root zone is related to the open water level in the ditches. This open water level is the most commonly used control variable in water management both in automated and in manual control mechanisms. The rise or fall of this level and the level itself have traditionally been the input signals for the decision to start/stop pumping, higher/lower crest levels. This still holds true for the modern automatically responding devices. The disadvantage of this kind of control mechanisms is that there is no straightforward relation between the open water level and the moisture content in the root zone. This moisture content is determined by the hydrological conditions in the preceding period, the soil characteristics and the drainage/infiltration conditions. In the situation where an operator has to take the decision to respond manually, this water level is not the only decision variable. His skill, his interpretation of the weather forecast and his knowledge of the drainage area and its population are all factors that play an important role in his decision. This control mechanism offers the best possibilities for refined control if the operator is very experienced. A problem may be caused by conflicting opinions of the inhabitants of the area about the operator's skill. Or about his judgement in the weighing of the conflicting interests of the different kinds of land use.

The control mechanisms applied differ in dependence of the local situation with respect to land use, soil characteristics, drainage conditions and availability and cost of drainage and water supply. Generally speaking it can be stated that under climatological conditions as prevail in the Netherlands there is a rainfall surplus in winter (October - March) and an evapotranspiration surplus in summer (April - September). In winter drainage is necessary and drainage conditions are optimized by lowering the surface water level. In summer evapotranspiration demand may exceed the water delivering capacity of the soil. To minimize shortages high surface water levels are created to stimulate sub-irrigation. In general the surface water levels will be chosen low during winter and high during summer under Dutch circumstances. Management of the surface water level may be accomplished automatically or manually and may use a wide range of control variables varying from open water levels to soil moisture values and groundwater levels or a combination of all. The control mechanism applied depends strongly on the physical and financial possibilities at hand.

In recent years several studies have been carried out in order to find a better, or in any case a more objectively based control mechanism. Here is mentioned the work of Lumadjeng (1985) and of Bakel (1986). Both authors use the groundwater level as the control variable in stead of the open water level. Lumadjeng (1985) uses the groundwater level as the control variable in his model based on the linear system theory. The objective function consists of a component for drought damage and damage due to too wet conditions and a component which accounts for erosion damage of the bank slopes due to too rapid changes of the open water level. He furthermore includes the weather forecast in his control system. Van Bakel describes a control method based on the optimal water supply to the root zone. He distinguishes four different control mechanisms, each related to the time of the year. In winter the groundwater level is the control variable, in spring he uses the groundwater level and the rise or fall of the groundwater level, during summer and in autumn the surface water level is related to the groundwater level and the water storage in the root zone. To prevent large differences in the target level of the surface water which may cause erosion damage to the bank slopes, manipulation possibilities for the weir are limited (Bakel, 1984; 1986; Walsum and van Bakel, 1983).

Operation rules for weirs and pumps applied in the agricultural areas in The Netherlands do not only depend on the optimal surface water level for arable or grassland areas. Important limitations in the higher and more sloping areas in the East and South regions are related to the fact that in summer water can only be supplied by pumping. The water control policy is aimed at conservation of water from February onwards in order to minimize the amount of water to be pumped up from the IJssel Lake, for reasons of the pumping costs and the quality of the water thus imported. In many cases the quality of the imported water is such that it may have a negative influence on the aquatic and terrestric ecology in the affected area. The control mechanism applied is further related to the groundwater drainage qualities of the soils and the workability limits of the soils.

In the low lying polders in the Western regions of the Netherlands the pumping costs are a important factor in the control mechanism. If electric pumps are installed pumping takes preferebly place during periods of the day when energy costs are low. To supply water costs are not the major concern, but in these areas much attention is given to the quality of the water thus imported. In these areas the control mechanism applied is further related to the storage possibilities and the drainage characteristics of the area. Storage possibilities are especially small in the peat areas where high surface waters are maintained to prevent settlement of the peat layers.

The flowchart of PRODU is designed in such a way that various water management policies can easily be incorporated. The open water level in PRODU is determined by the:

. precipitation and evaporation directly on/from the open water surface.
. drainage of other than arable areas, such as paved areas. This can be urban area, parking lots or glasshouses for example.
. infiltration and drainage to and from the arable land.
. seepage/leakage.
. water supply from external sources.

An important assumption for the application of PRODU and for the interpretation of the results refers to the determination of the open water level in the model. The open water in the area under study, the channel system, is assumed to be so dimensioned that hydraulic head differences needed to transport the water are negligible. This holds true in most cases if the areas are not too large in relatively flat territory such as in the western part of the Netherlands. For example, level differences in discharge periods in the new Flevoland polders are about 2-3 cm because these channels are designed for navigation. In the older polders in the Westland, level differences are less than 5 cm because the total length of the main canal is fairly short. This assumption simplifies the determination of the level to the simple balancing of in- and outflowing quantities of water.

The following drainage options are available:

- An automatically responding pumping station which starts or stops pumping in reaction to surface water level sensors. Distinction can be made between summer and winter "on" and "off" levels.
- A weir/sluice with constant downstream water level which crest height may be altered for summer and for winter.

Other control mechanisms where groundwater levels or moisture content at a certain depth can easily be incorporated. It has not been made a point for research in this thesis to elaborate on them.

The water supply options are:

- Discharge of an adjacent polder with similar properties, achieved with a pumping station with summer and winter "on" and "off" levels.
- Discharge of an adjacent polder with similar properties, achieved with a weir with summer and winter level.
- Water supply controlled by means of a pump for a given period and capacity responding to the open water level of the polder.

2.2.4 Workability of the soil

A few times a year the top soil has to meet some requirements with respect to bearing capacity and seedbed preparation. In the periods given in Fig. 2.5., access to the soil is required for the following cultivation activities: preparing the sowing bed; sowing or planting; spraying; harvesting; stubble cultivation and ploughing. Delay or inadequate execution of one of these cultivation activities may lead to a decrease in yield (Fig. 2.6) or to deterioration of the structure of the soil, thus possibly affecting future crops. The major constraint for adequate execution with respect to bearing capacity and to seedbed preparation is the moisture content of the top soil. The most critical periods in Dutch circumstances are spring and autumn, because the cultivation activities that take place in the period from May to the beginning of September are not important and seldomly disturbed by too wet or too dry conditions of the soil. Under normal drainage conditions, the conditions for ploughing in November are fulfilled. If the groundwater reservoir is filled and if the unsaturated profile above the groundwater table is at field capacity, as is the situation in spring, the workability limits can be complied with more easily if drainage is stimulated by establishing lower (winter) surface water levels. In spring when the seedbed is prepared, it is of great importance that the actual moisture content of the topsoil is within the range of soil moisture contents suitable for tillage, which is rather narrow for clay for example. If the soil is too dry, it will be difficult to crumble, while crumbling will take more time in a period when workable days are scarce. If the soil is too wet, it is liable to smearing and puddling, and its structure may be seriously disturbed. The best consistency for working the soil is the friable state, which is below the lower plastic limit (LPL-value) (Smedema,1979). The lower plastic limit is the moisture content at which the soil consistency changes from friable to plastic. In autumn, at the time of the potato and sugar beet harvests, the same narrow range of soil moisture contents makes the soil suitable for harvesting. If the soil is too dry, clods will disturb the transport in the potato lifter.

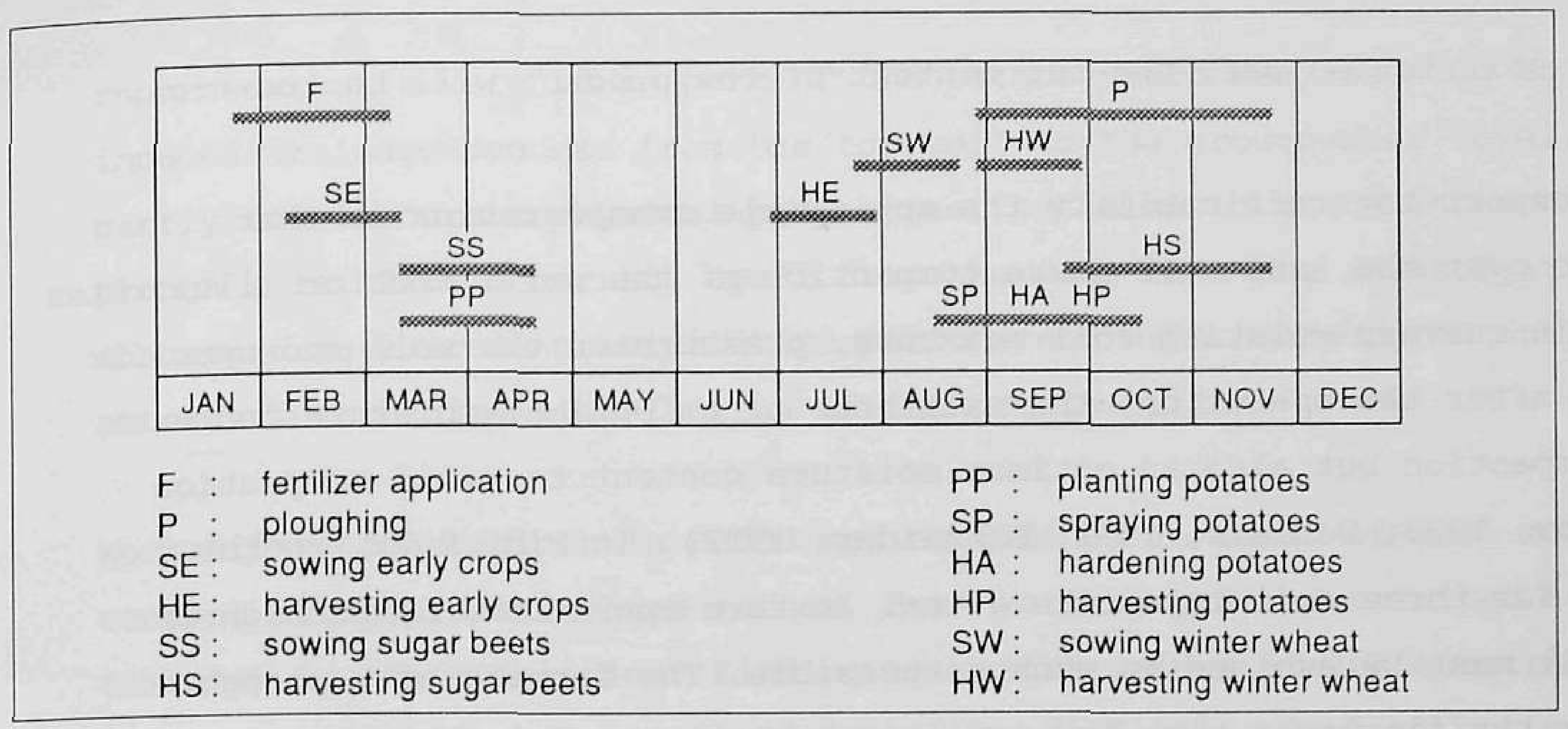


Fig. 2.5. Time schedule for cultivation activities, over a one year period.

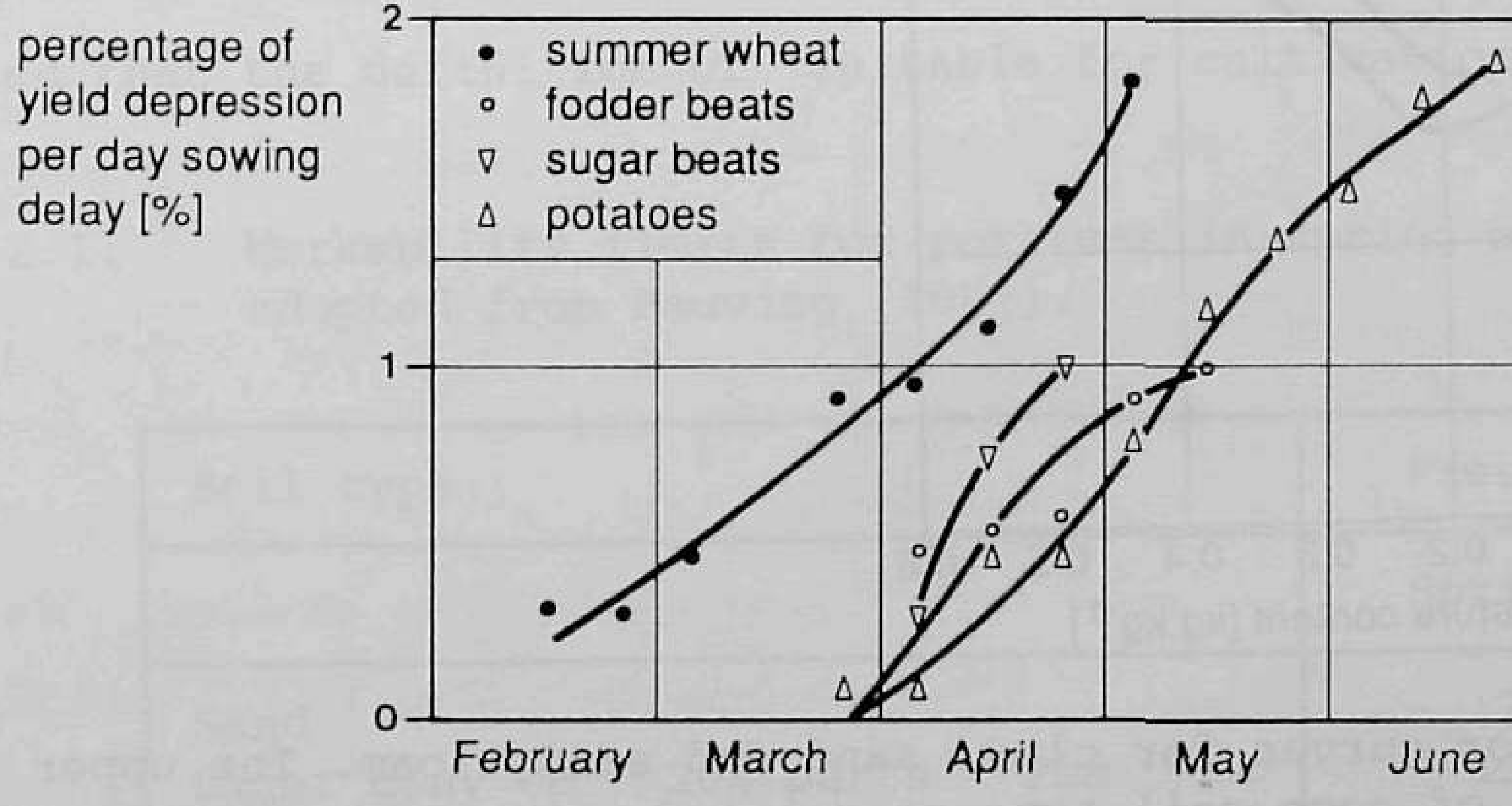


Fig. 2.6. Yield depression for summer grains and sugar beet as a result of delay in sowing (Wijk and Feddes, 1981, derived from Wind, 1960).

If the soil is too wet, the tar content of the product will be too high.

With respect to trafficability the applicable consideration is that traffic over the land will cause compaction of the soil. Proctor illustrates with his curves, relating soil moisture, pressure on the soil and specific weight after the operation, the existence of an optimum moisture content for compaction but also an optimum moisture content to avoid compaction (Proctor, 1933; Daniels, 1977; Schneider, 1982). In Fig. 2.7, Proctor curves for three soil types are given. In farm operation, compaction of the soil must be avoided as much as possible. The forming of a plough sole or traffic layer with low infiltration rates and low hydraulic conductivity will decrease the effectiveness of the subsurface drainage system in a drainage situation, resulting in fewer workable days and therefore lower yields. During the growing season the same plough sole may cause decreased capillary rise because of the lower hydraulic conductivity. It will also affect root development causing the crop to be more sensitive to drought. Compaction is increased if the pressure on the soil is repeated. This fact, and taking into consideration the pressure distribution in the soil under wheel loading (Söhne, 1953), the moisture content profile and the above Proctor curves, it is evident that a plough sole is easily formed. The economic consequences of the presence of a plough sole were clearly shown in Zeeland, the South-Western province of the Netherlands, in 1974. Boels and Wind (1975; a, b) reported that a very large part of the potato yield was lost due to the impeded drainage process from the top soil to the groundwater level, partly due to normal physical properties, but also to the existence of a plough sole. This occurred, apart from the fact that the investigation concerned an extreme precipitation period, not with respect to total amount, but to time distribution of the precipitation.

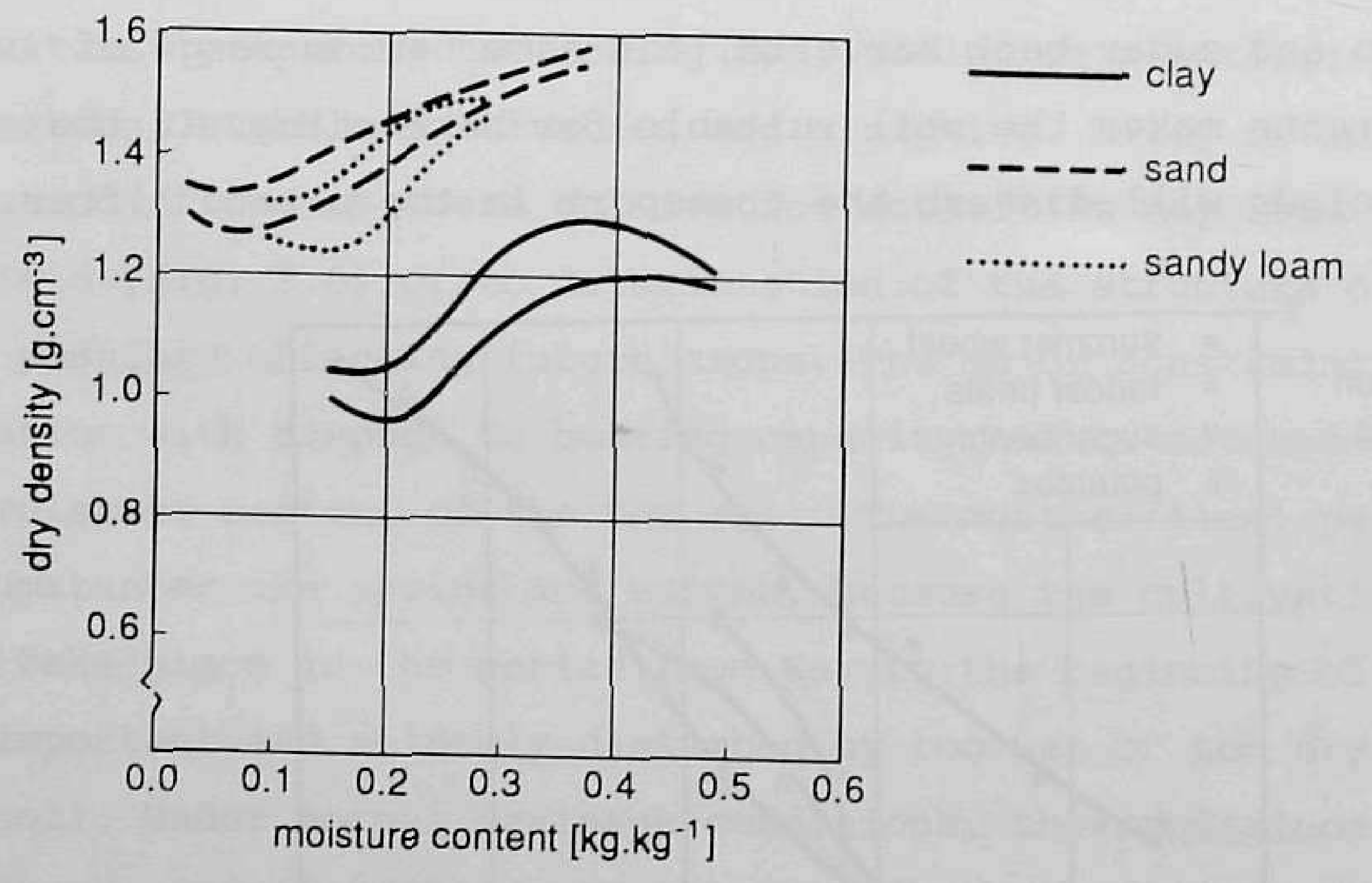


Fig. 2.7. Proctor curves for clay, sand and sandy loam. The upper curve of each soil type corresponds to a pressure of 1.0 kg.cm^{-2}, the lower curve to a pressure of 0.5 kg.cm^{-2}. (Schneider, 1982).

Workability constraints in PRODU may be defined in terms of soil moisture content or in pressure head values. Both variables are determined every timestep in the simulation. Finding the right values of the constraints for workability for use in PRODU is rather difficult. Several options with respect to workability limits have been proposed. Perdoc and Tanis (1975) suggest the pressure head values pF = 2.0 on the wet side and pF = 2.7 on the dry side for light cultivations as workability limits. These limits have been verified against data of Hokke, who, being an extension officer, laid down his judgement about the workability conditions over a period of 30 years. Another approach was adopted by Beuving (1982): in order to find the workability limits he asked the farmers in a research area to give their opinion on the workable conditions of their soils. At the same time he measured the pressure head at two depths in the soil. According to the farmers the cultivation activities can be accomplished within these limits without detriment to the soil structure. These limits are given in Table 2.1. for potatoes. They are represented by the pressure head values of the soil at 5 cm below surface in spring, and 15 cm in autumn.

The most important criteria for the farmers were the operation of the screen of the potato harvester and the degree of rutting. Beuving reported that the definition of "suitable for cultivation" depended on

Table 2.1. Workability limits for potatoes in spring and autumn, adopted from Beuving (1982).

Soil type	Pressure head [cm]	
	Spring	Autumn
Sand	- 70	- 60
Light clay 8 - 20% parts < 2μm	- 100	- 100
Light clay 20 - 40%	- 120	- 120
Clay > 40%	- 80	- 180

what soil the farmer owned. A farmer on heavy clay soils is likely to accept a higher tar content than his colleagues on other soils. Besides, there is a tendency for farmers to apply their limits more stringently in spring than in autumn. To what extent these limits are generally applicable is debatable. Simply applying a pressure head value does not take into account the influence of the content of organic matter. The LPL-value increases with the organic matter content of the soil. The soil can then be workable at higher moisture contents than corresponding to pF 2.0 (Smedema, 1979). Schneider (1982) showed that lighter soils are easy to compact at lower moisture contents. However, Schneider does not give the pF-curves. Assuming the soils to be more or less "standard" sand, loamy sand and clay, the optimum values reported by Schneider to avoid compaction would correspond to approximately pF = 2.0 for sand up to pF = 3.0 or higher for clay. Feddes and van Wijk (1976) reported a lower limit of pF 2.0 for sowing summer wheat as favourable working conditions and a lower limit of pF = 2.5 for sowing sugar beets and planting potatoes.

The sensitivity of the model for this criterion is based on the fact that the moisture content does not fluctuate very much over this period. This is caused by low potential evapotranspiration values due to the low radiation values in spring. The actual soil evaporation is a fraction of the potential evapotranspiration, and the transpiration is as yet absent. This low demand from the atmosphere and the fact that the soil profile is at field capacity result in very small to negligible fluctuations of the moisture content in the root zone. In consequence of this absence of fluctuation, a small change in the workability limit will result in either a very early planting/sowing date or a very late date.

It should be realized that the workability limit is applied to a simplified model of the complex real soil-water-plant system. It does not take account of cracks in the soil, nor of the fact that the surface is ploughed in November and then, on freezing, breaks up into smaller clods. The actual evaporation of the bare soil can then deviate substantially because the actual evaporating surface is greater than accounted for in the model. This can be partly corrected by adopting a higher hydraulic conductivity for the top layers of the top soil, but that does not really solve the problem. Another approach is to apply a model LPL value, somewhat lower than the moisture content at pF 2.0 and to check the results over 30 years.

In the absence of real data, the method which is adopted in PRODU is to choose the LPL value in such a way that reasonable yields are obtained in all of the 30 years that are considered in all simulations in Chapter 3 for a specific soil. This is an acceptable method as long as this value is not changed during the evaluation of different variants that are under study. If the same value is chosen for the different variants, it will not affect the outcome of the comparison. The day on which the total of the workable days, determined in this manner, equals the required number of workable days, is taken as planting day. The required number of days is in all simulations described in this thesis taken equal to 5.

2.2.5 Emergence date

Feddes found from information in literature and from field and laboratory experiments with various crops (radish, spinach, beans and garden beets), that "heat sums can give a relatively accurate prediction for the emergence date if soil moisture is taken into account" (Feddes, 1971). In conformity with Hunter and Erichson (1952), he considers the first five days to be the most important for swelling, initial water uptake and recommencing of metabolism of the seeds and thus the best indicator for the determination of the emergence date. With data from experimental fields concerning sowing, planting and emergence dates, daily temperatures and precipitation, relations between these parameters are determined for

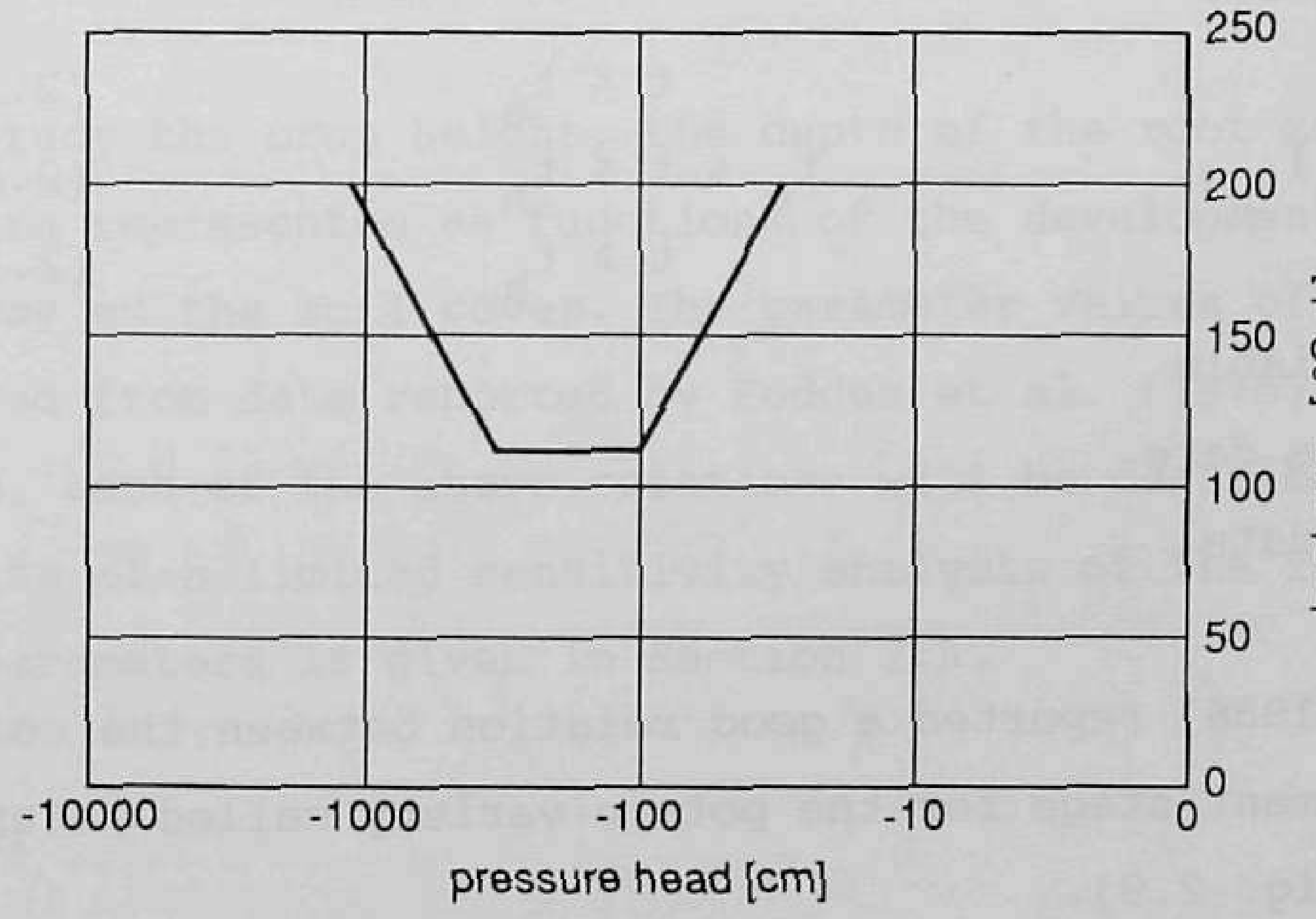


Fig. 2.8. Relation between the average pressure head of the first five days after planting and the minimal necessary heat sum for potatoes to emerge (Wijk et al.,1988).

cereals, potatoes and sugar beets. In Fig. 2.8 the relation is given for potatoes. It shows that the required heat sum, expressed in degree-days, increases proportionally as the pressure head deviates more from the optimal pressure head interval. This relation is selected for use in PRODU. The routine concerned determines the heat sum from the planting date and the date on which the heat sum equals the required heat sum corresponding the average pressure head, is taken as emergence date.

2.2.6 Development of the crop

To simulate the development of the crop as determined by transpiration, root extraction rates and the development of tubers, the following parameter values must be known a priori or must be generated in the programme: the depth of the root zone, the crop height (Monteith, Rijtema) or crop factor (Penman), the soil cover, the leaf area index and the ratio of the harvested part of the plant (tubers) to foliage and roots (β-ratio) during the year.

However, because the emergence date is not known, the above values cannot be established a priori. For a crop such as potatoes it can be stated that the harvest date is more or less fixed and known, depending on local tradition and crop variety. In that case the development stage of the plant, being a function of the harvest date, the emergence date and the actual date, can be determined during the simulation. The development stage (D_s) is defined as:

$$D_s(t) = 0.0 \qquad t < t_e \qquad (2.12)$$

$$D_s(t) = (t-t_e)/(t_h-t_e) \qquad t_e \leq t \leq t_h \qquad (2.13)$$

$$D_s(t) = 0.0 \qquad t > t_h \qquad (2.14)$$

where: t = time variable

t_e = emergence date

t_h = harvest date.

Van Wijk and Feddes (1986) reported a good relation between the soil cover and the development stage for the potato variety called Astarte over several years (Fig. 2.9).

The leaf area index can be related to the soil cover as (Schans et al., 1984):

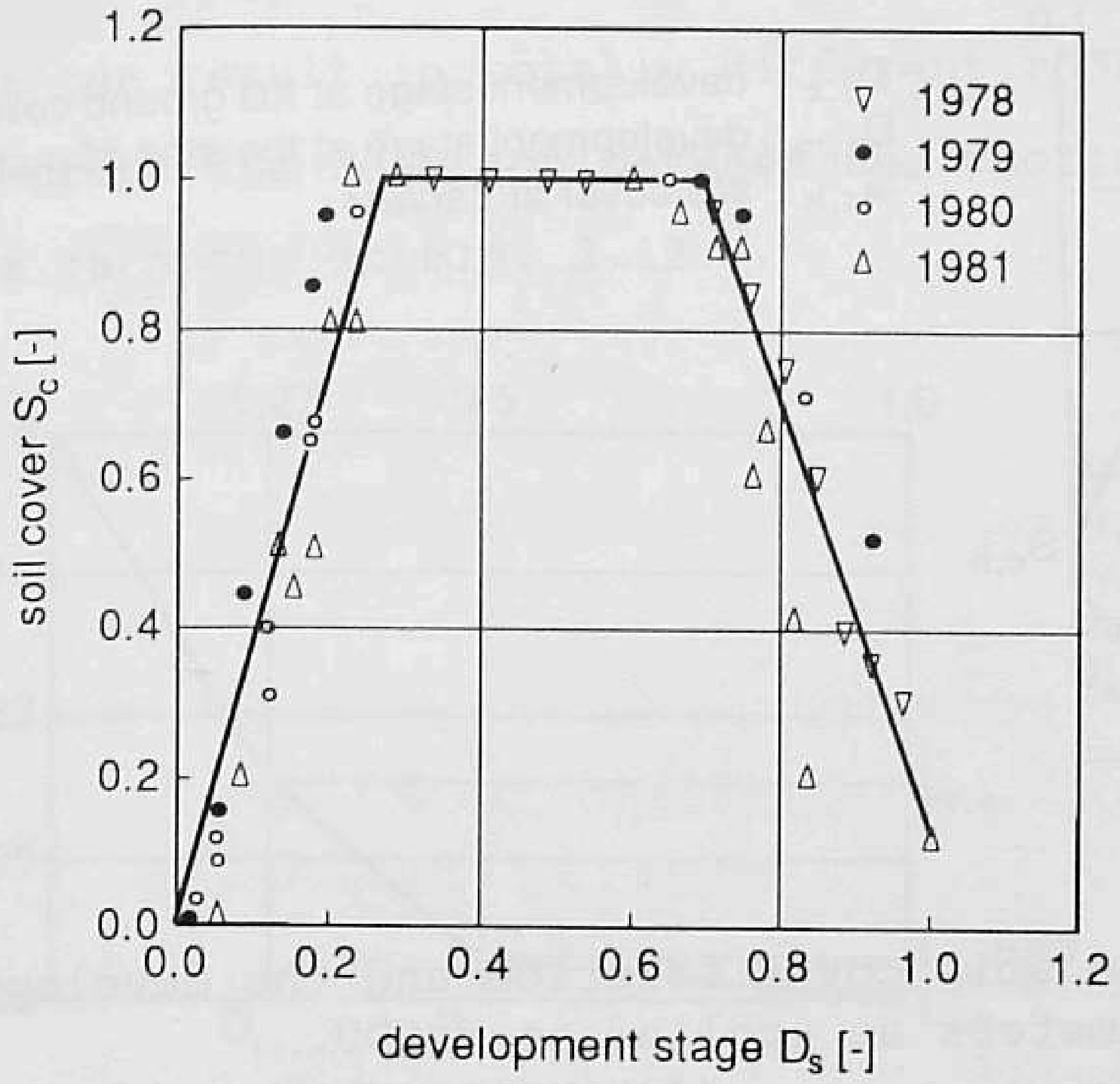


Fig. 2.9. Relation between the soil cover and the development stage, for various years, for the variety Astarte grown at different locations over a number of years (Van Wijk and Feddes, 1986)

$$I_L(t) = 2.6 \, . \, S_c(t) + 15 \, . \, S_c^2(t) + 0.9 \, . \, S_c^3(t) \qquad (2.15)$$

where: L_i = the leaf area index at time = t

S_c = the soil cover at time = t.

This solves the problem for two of the five parameters mentioned at the beginning of this section. It seems logical to follow the reasoning used for the soil cover equally for the other three parameters, crop height, rooting depth and β-ratio. FAO gives crop factors related to the development stage for various crops (Doorenbos and Kassam, 1979).

In this study the crop height, the depth of the root zone and the β-ratio are represented as functions of the development stage in a similar way as the soil cover. The parameter values of these functions are derived from data reported by Feddes et al. (1978). In the following, each of the above relations will be described in more detail. The results of a limited sensitivity analysis of the relations for the various parameters is given in Section 2.3.

Soil cover

The relation between the soil cover and the development stage is given in Fig. 2.10.

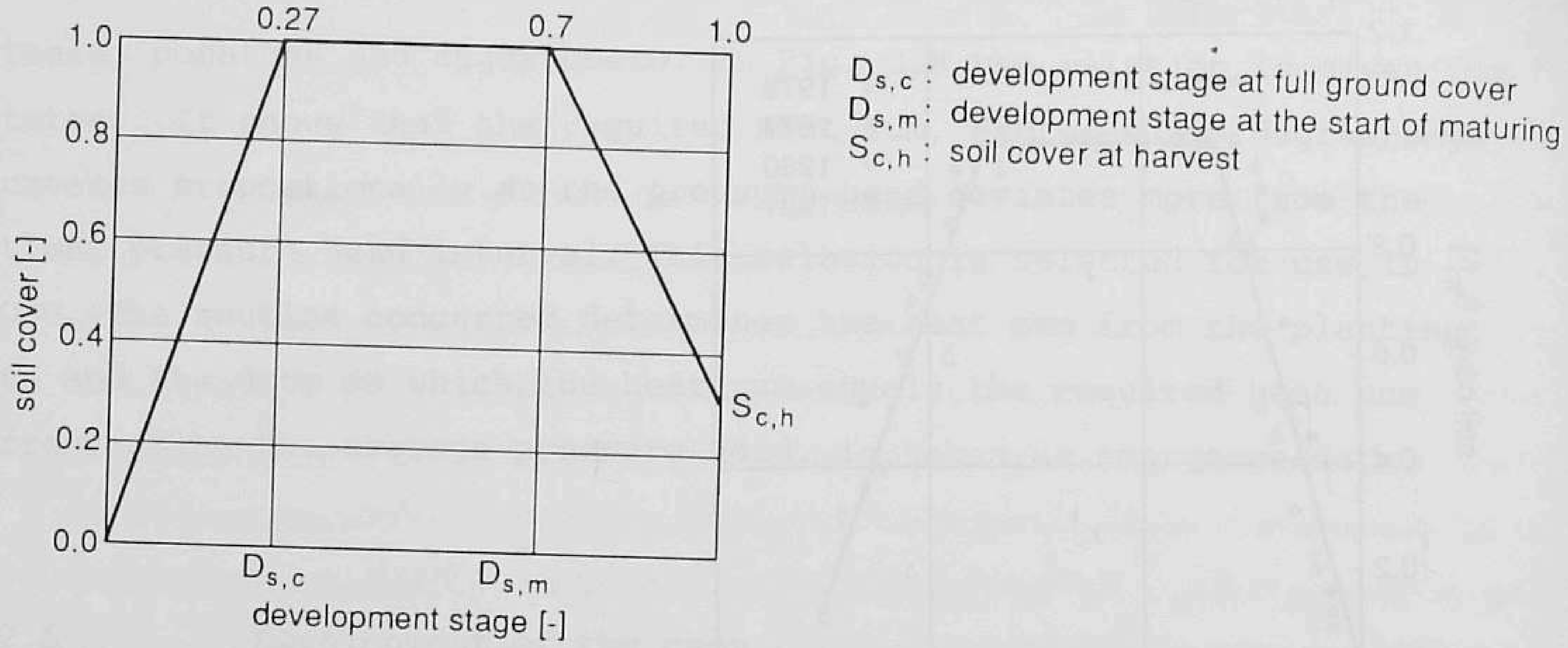


Fig. 2.10. Relation between the soil cover fraction and the development stage with the parameters as applied in PRODU.

Crop height

The relation between the crop height and the development stage is given in Fig. 2.11.

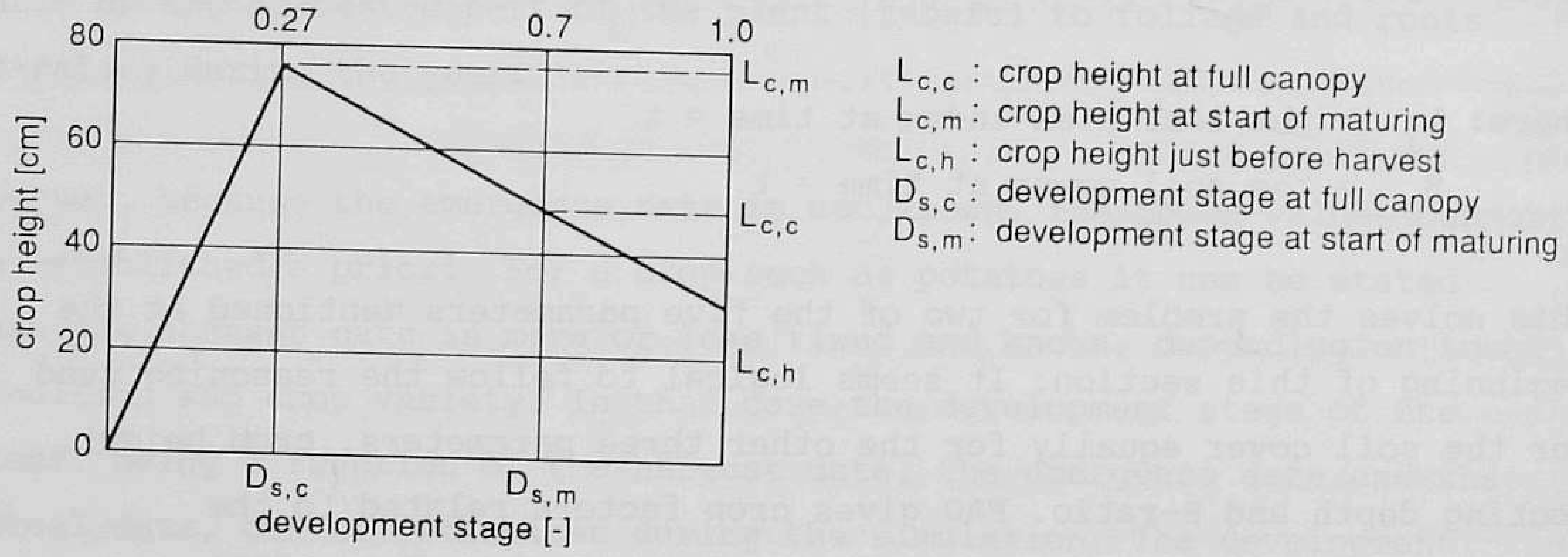


Fig. 2.11. Crop height in relation to the development stage with the parameters as applied in PRODU.

Depth of the root zone

According to Lieshout (1960) the rooting depth of a crop depends strongly on the rooting capacity and rooting type of the crop on the one hand and the soil qualities, the profile and the groundwater levels on the other hand. The presence of a layer of greater density (plough sole) or a change from clay to sand can stunt the root development. Intensive and deep cultivation promotes root development in depth as well as in density by the formation of more root branches. The great variation in soil qualities and profiles and the strong modification abilities of the roots can result in totally different rooting patterns for one and the same crop. The relation between the rooting depth and the development stage is shown in Fig. 2.12.

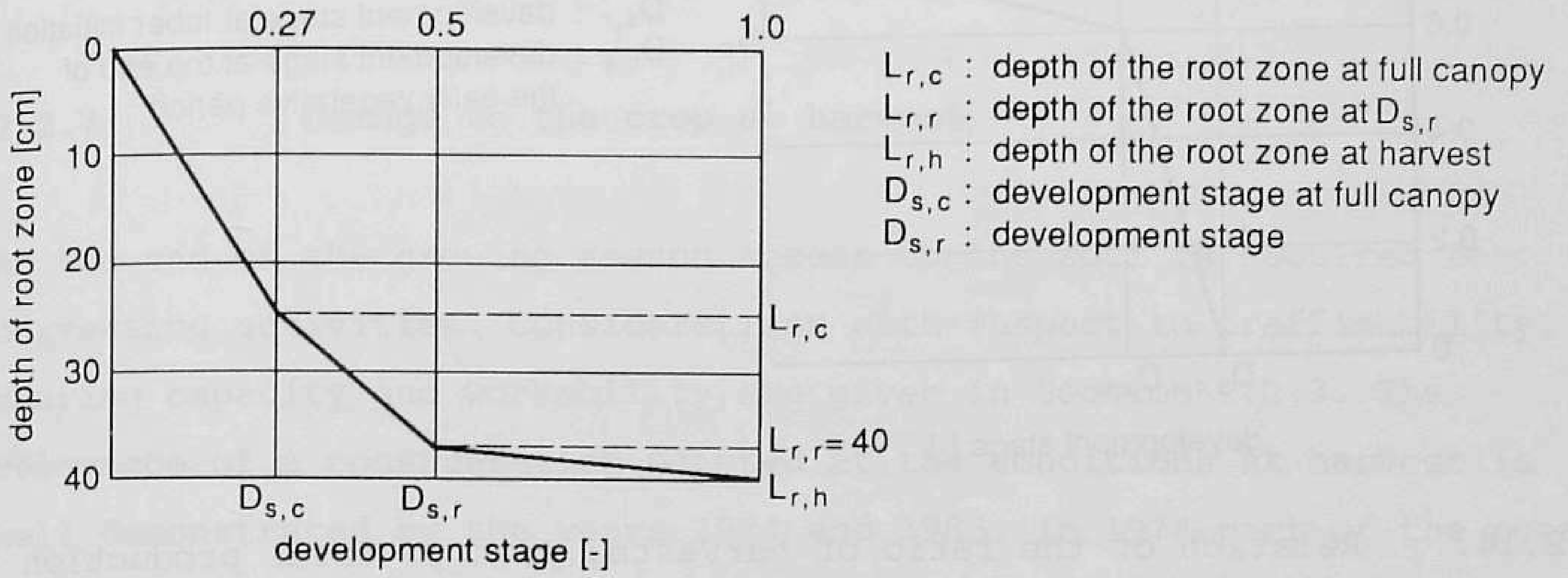


Fig. 2.12. Rooting depth in relation to the development stage with the parameters as applied in PRODU.

Ratio of harvested part and total production

In order to determine the yields of tubers from the dry matter production it is necessary to apply a factor β, the ratio of harvested part over total plant production. This ratio varies considerably with time, as Fig. 2.13 shows. Generally speaking, this factor is zero in the period up to tuber initiation, i.e., the first 30 to 40 days, to increase strongly in the early vegetative period and somewhat less strongly up to harvest. The development of the value of this factor depends, among other factors, strongly upon the meteorological conditions, the crop

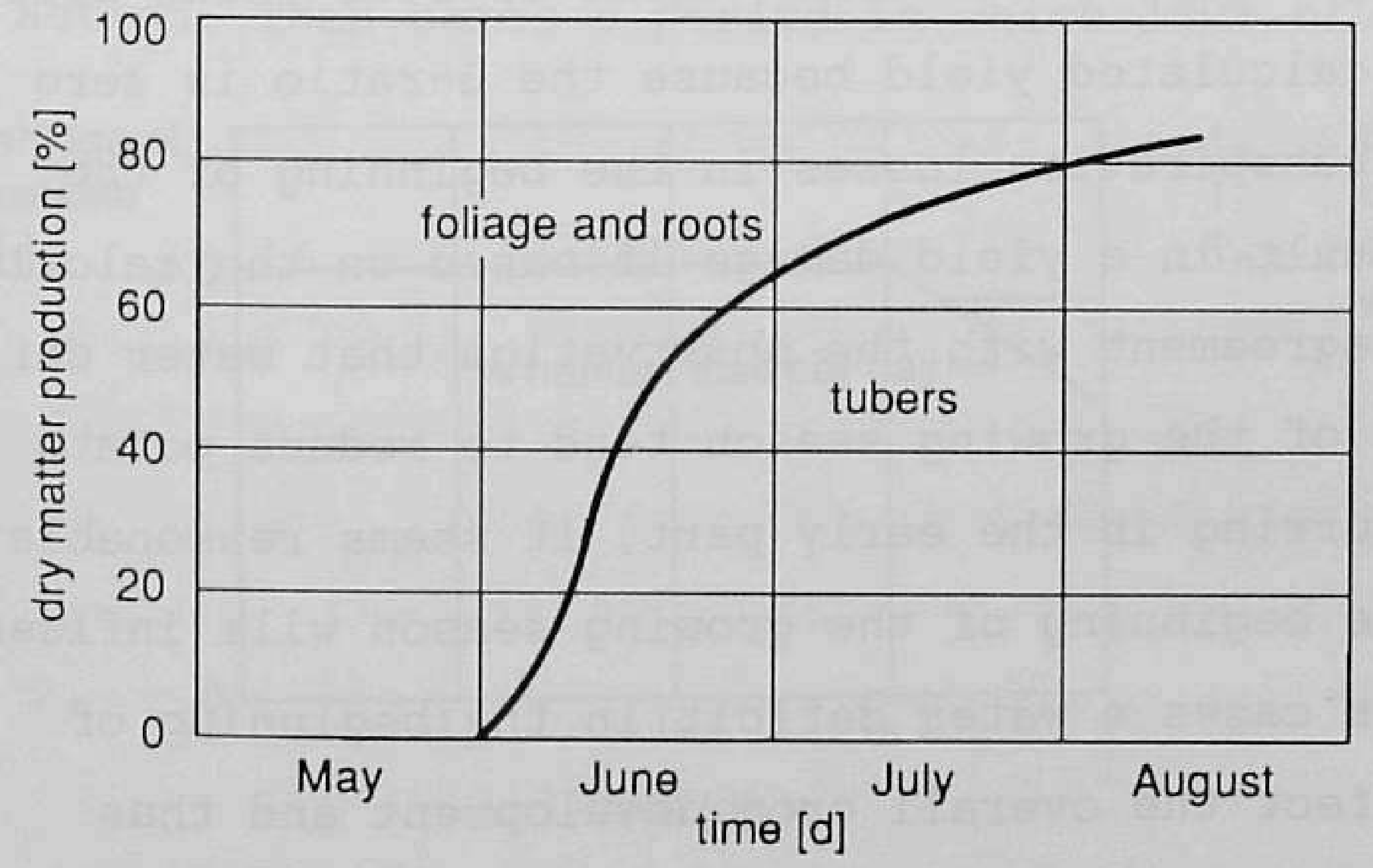


Fig. 2.13. Distribution of foliage and root production against tuber production during the year 1976 (Feddes et al., 1978).

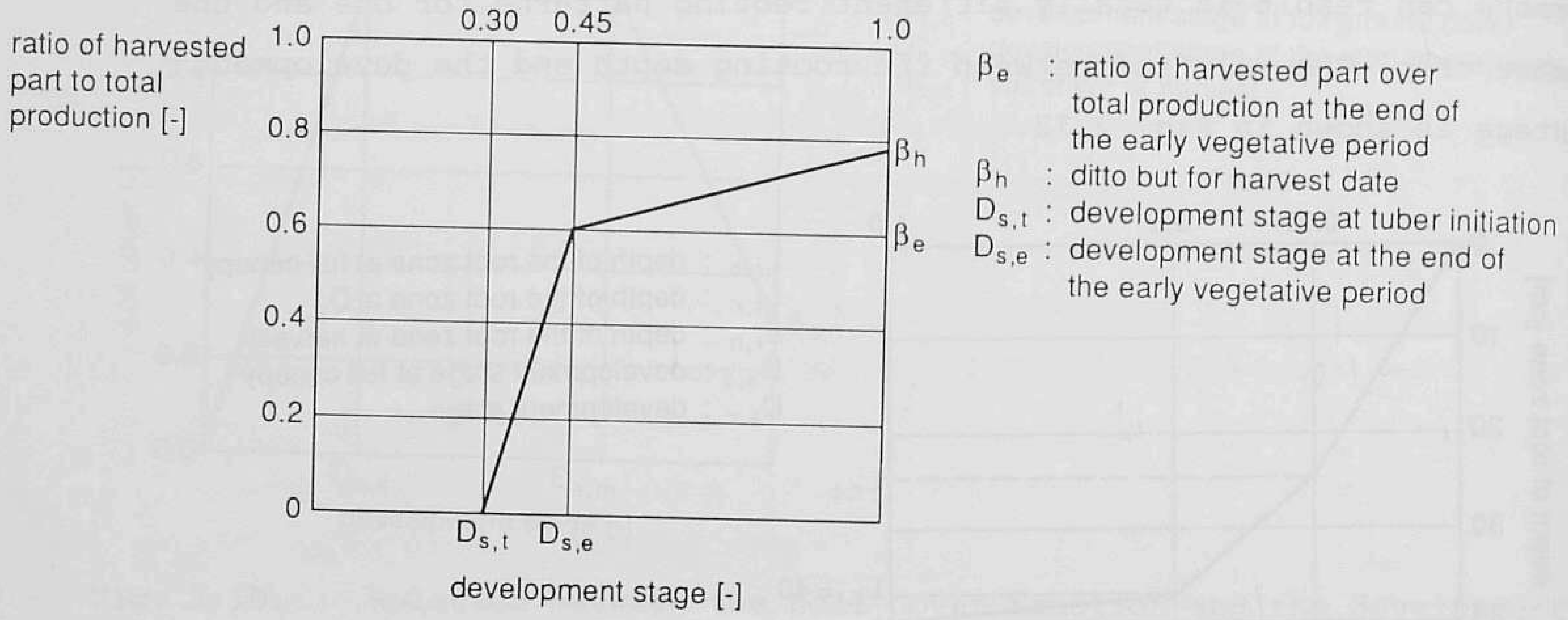


Fig. 2.14. Relation of the ratio of harvested part to total production with the development stage, as applied in PRODU.

variety, the drainage conditions, the availability of irrigation water and the fertilizer supply (Mosz and Kowalik, 1982). Under Dutch circumstances an optimal fertilizer application may be assumed. The relation between the β-ratio and the development stage as applied in PRODU is given in Fig. 2.14.

For the interpretation of the final results of a simulation with PRODU it should be realized that this β-ratio also acts as a weighting factor to the transpiration deficiencies during the growing season. Yield damages which are expressed in the transpiration deficiency only, may differ from yield damages which are based on the ratio of calculated actual and potential tuber yield. The extent of the difference depends on the meteorological and drainage circumstances. A transpiration deficiency of a certain magnitude in the first three or four weeks will not result in any loss of calculated yield because the β-ratio is zero in that period. Although transpiration losses in the beginning of the growing season will not result in a yield damage if based on the calculated potato yield, which is in agreement with the observation that water deficits in the middle to late part of the growing season tend to reduce potato yields more than those occurring in the early part, it seems reasonable that a water deficit in the beginning of the growing season will influence the final yield. In serious cases a water deficit in the beginning of the growing season will affect the overall crop development and thus also affect the values of soil cover and crop height. It is for this reason that in the presentation of the final results of a simulation with PRODU the losses are given in terms of potato yield but also in terms of total dry matter yield, calculated with a value of 1.0 for the β-ratio.

2.2.7 Damage to the crop at harvest

At the end of the growing season access to the soil is required for harvesting activities. Considerations with respect to trafficability, bearing capacity and workability are given in Section 2.2.3. The relevance of a consideration pointed at the conditions at harvest is well demonstrated by the years 1974 and 1983. In 1974 part of the potato yield in the Netherlands, and for some farmers a considerable part, was lost due to too wet conditions at harvest. Though on a much smaller scale the rainfall in 1983 was the cause of the difficulties in harvesting potatoes and sugar beets. This harvest period deserves a special consideration because besides the workability aspect, there is the aspect of a loss of quality and quantity of the potatoes due to an unusually long period under very wet conditions. After a very wet period in which the potato ridges have lost their shape the potato harvester will pick up a smaller percentage of the total yield. Under these circumstances the potatoes will be at risk of greater blue spot susceptibility. The longer such a period lasts the more serious these aspects become. Quantitative data on the above aspects are not available, but with the autumns of 1974 and 1983 fresh in mind some approximation can be made. Therefore the approach outlined in Fig. 2.15 is followed here. Spraying of the leaves is followed by a period without any action. Then comes a period in which 100% of the yield can be

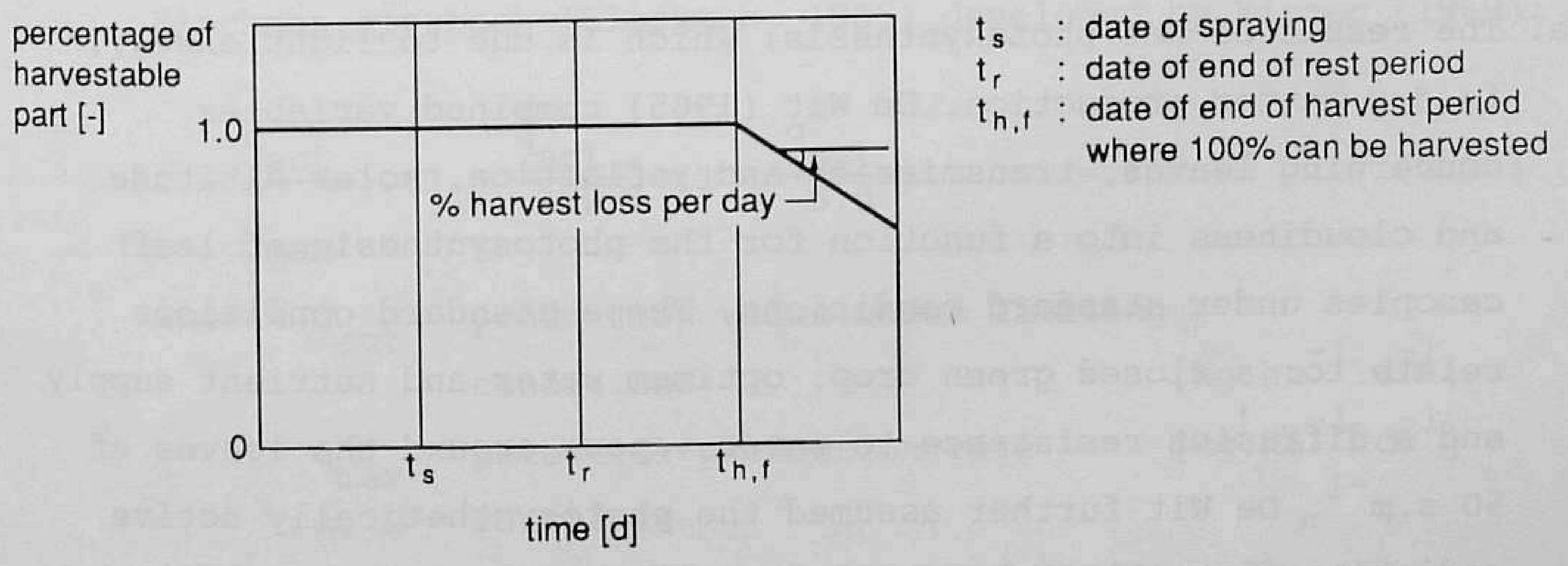


Fig. 2.15. Schematisation of the potato yield losses due to too wet conditions over too long a period with the parameters as applied in PRODU.

harvested. The second period is followed by a period in which a diminishing part of the yield can be harvested. This approach does not obviate the fact that under these circumstances the farmers tend to be less careful with respect to workability and trafficability limits (Beuving, 1982). Simulation is possible by adjusting the workability limit after a certain period. This has been rejected for three reasons, however. Firstly, in the period 1951-1980, for which most simulations have been carried out, it was only in 1974 (fairly often) and in 1954 and 1968 (rarely) that part of the crop was not harvested because of the too wet conditions. Secondly, insufficient data are available. Thirdly, it will be very difficult to simulate under what conditions the help of the army, students or schoolchildren will be called in.

2.2.8 Crop yield

The last step in PRODU is the determination of crop yield. For this the transpiration as calculated with SWATRE must be converted into crop yield. For the determination of the potential and actual dry matter and of the potential and actual potato yield, the model of Feddes et al. (1978), called CROPR, was adopted. The crop yield is determined in CROPR in three steps:

a. Determination of the potential production of a standard crop under standard conditions.

b. Determination of the potential production of a potato crop when there are no limiting production factors.

c. Determination of the actual production of a potato crop if water is considered as the only limiting production factor.

Re. a. The result of net photosynthesis, which is due to light energy, is dry matter production. De Wit (1965) combined variables concerning leaves, transmission and reflection, solar altitude and cloudiness into a function for the photosynthesis of leaf canopies under standard conditions. These standard conditions relate to: a closed green crop, optimum water and nutrient supply and a diffusion resistance to water vapour around the leaves of 50 $s.m^{-1}$. De Wit further assumed the photosynthetically active radiation flux (R) to be equal to half the short-wave radiation, and the average photosynthetically active radiation flux on overcast days (R_o) to be equal to 20% of the solar radiation flux involved in photosynthesis on clear days (R_c). The gross potential growth rate (P_{st}) is calculated from:

$$P_{st} = G.P_o + (1 - G).P_c \qquad (2.16)$$

where: P_{st} = gross potential growth rate $[kg.ha^{-1}.d^{-1}]$

P_c = gross growth rate for clear days $[kg.ha^{-1}.d^{-1}]$

P_o = gross growth rate for overcast days $[kg.ha^{-1}.d^{-1}]$

G = mean fraction of the time that the sky is overcast, calculated as: [-]

$$G = (R_c - R) / (0.8.R_c) \qquad (2.17)$$

De Wit provide tables listing values of R_c, P_c and P_o as a function of the latitude and the time of year.

Re. b. To arrive at potential dry matter production of a potato crop under ideal conditions ($\dot{q}_{max}$), the influence of temperature on plant growth (α_t), soil cover (Sc) and respiration losses (φ_r) must be taken into account, giving:

$$\dot{q}_{max} = P_{st}.\varphi_r.\alpha_t.S_c \qquad [kg.ha^{-1}.d^{-1}] \quad (2.18)$$

For the potential potato production the ratio of harvested part to total production (β) must be included:

$$\dot{q}_{max} = P_{st}.\varphi_r.\alpha_t.S_c.\beta \qquad [kg.ha^{-1}.d^{-1}] \quad (2.19)$$

Re. c. For a general production function Feddes et al. (1978) chose a Blackman approach (Blackman, 1905) developed by Visser (1969):

$$(1 - \frac{\dot{q}_{act}}{\dot{q}_{max}}).(1 - \frac{\dot{q}_{act}}{A.w}).(1 - \frac{\dot{q}_{act}}{B.u})...... = F \qquad (2.20)$$

where: $\dot{q}_{act}$ = production value under limiting conditions $[kg.ha^{-1}.d^{-1}]$

$\dot{q}_{max}$ = potential production rate $[kg.ha^{-1}.d^{-1}]$

A, B = constants

w, u = production factors such as water, fertilizer,

F = mathematical factor [-]

The terms A.w and B.u describe the effect of the production factors water and fertilizer on production. If water is considered to be the only limiting production factor, equation (2.20) becomes:

$$\left(1 - \frac{\dot{q}_{act}}{\dot{q}_{max}}\right).\left(1 - \frac{\dot{q}_{act}}{A.w}\right) = \xi \qquad (2.21)$$

where: ξ = a mathematical flexibility constant close to zero.

The general shape of this production function is shown in Fig. 2.16. Equation (2.21) is a non-rectangle hyperbole with two asymptotes of the left indicates the productivity of a crop for water that is well supplied with nutrients, the upper represents the production level limit determined by other factors than the water supply.

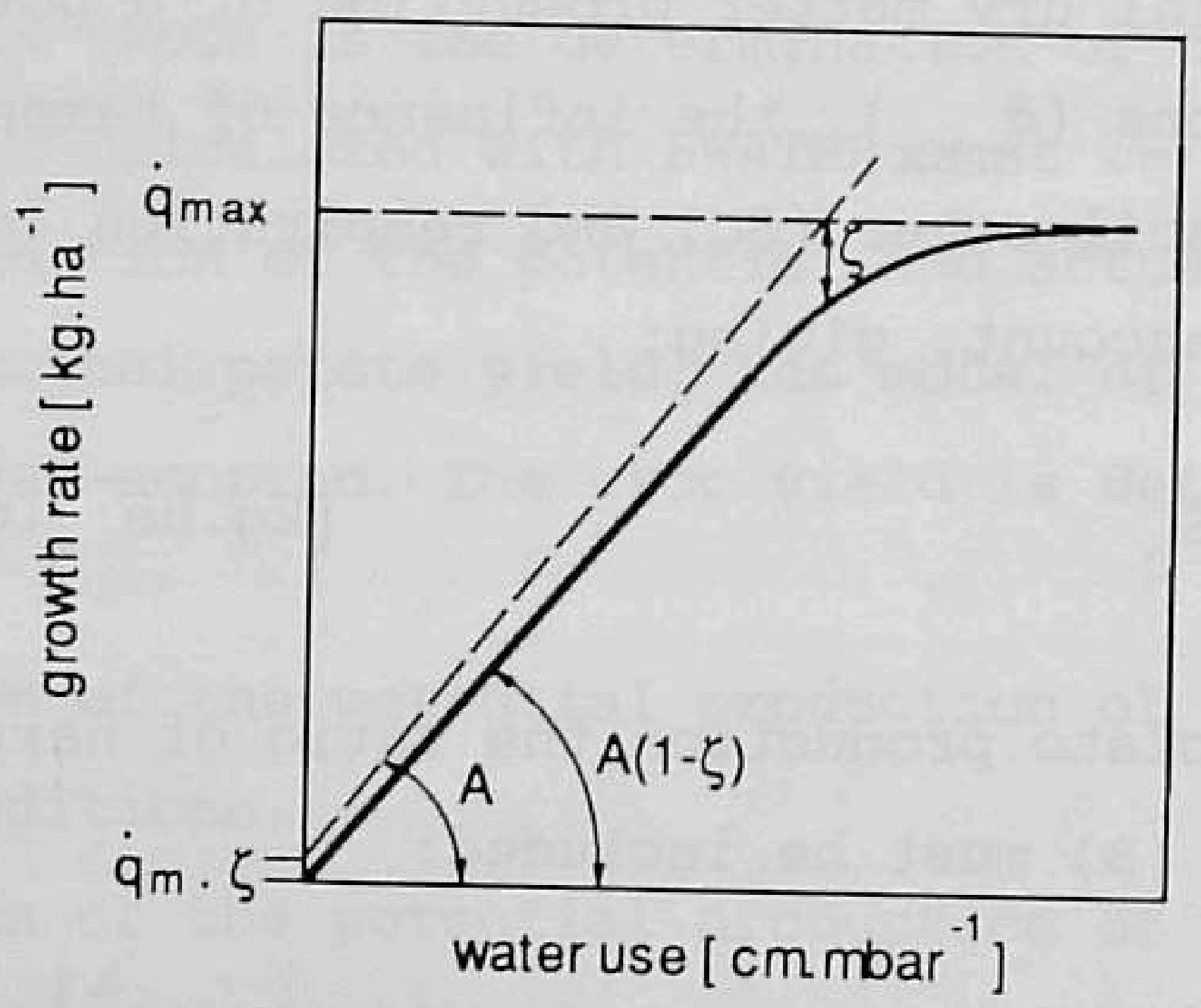


Fig. 2.16. Visser's growth function (1968).

The factor A.w represents the dry matter production determined by transpiration. De Wit (1958) found the ratio of total crop yield in kg.ha^{-1} to total transpiration in mm to be a constant which depends on climatological conditions. Bierhuizen and Slayter (1965) ascribed the relation between this constant and the climatological conditions to the vapour pressure deficit of the air and found:

$$\dot{q} = A \, . \, \frac{E_t}{\Delta e} \qquad (2.22)$$

where: $\dot{q}$ = growth rate [kg.ha^{-1}.d^{-1}]
E_t = transpiration rate [mm.d^{-1}]
Δe = vapour pressure deficit [mbar]
A = maximum water use efficiency [kg.ha^{-1}.mm^{-1}.mbar^{-1}]

The constant A is found not only to differ for different varieties but also for different meteorological and environmental conditions (Slabbers et al., 1979). Replacing w in equation 2.21 gives:

$$\left(1 - \frac{\dot{q}_{act}}{\dot{q}_{max}}\right) . \left(1 - \frac{\dot{q}_{act}}{A \frac{E_t}{\Delta e}}\right) = \xi \qquad (2.23)$$

Solving this equation, the actual and potential growth rate for day i of the growing periods is:

$$\dot{q}^i_{act} = \frac{A}{2}\,\frac{E^i_t}{\Delta e^i} + \frac{\dot{q}^i_{max}}{2} - \frac{1}{2}\left[\left(\dot{q}^i_{max} + A.\,\frac{E^i_t}{\Delta e^i}\right)^2 - 4.\dot{q}_{max}A \, . \, \frac{E^i_t}{\Delta e^i}\,(1-\xi)\right]^{\frac{1}{2}} \qquad (2.24)$$

$$\dot{q}^i_{pot} = \frac{A}{2}\,\frac{E^i_{tp}}{\Delta e^i} + \frac{\dot{q}^i_{max}}{2} - \frac{1}{2}\left[\left(\dot{q}^i_{max} + A.\,\frac{E^i_{tp}}{\Delta e^i}\right)^2 - 4.\dot{q}_{max}A \, . \, \frac{E^i_t}{\Delta e^i}\,(1-\xi)\right]^{\frac{1}{2}} \qquad (2.25)$$

The final potential and the actual yield can be calculated as the sums of the daily productions in the growing period:

$$Q_{act} = \sum_{i=1}^{n} \dot{q}^i_{act} \qquad (2.26)$$

$$Q_{pot} = \sum_{i=1}^{n} \dot{q}^i_{pot} \qquad (2.27)$$

2.3 **Sensitivity analyses**

In this study the crop height, the depth of the root zone and the β-ratio are represented as functions of the development stage in a similar way as the soil cover. The parameter values of these functions are derived from data reported by Feddes et al. (1978). In the following, each of the above relations will be given in more detail, as well as the results of a limited sensitivity analysis of the relations for the various parameters. This sensitivity analysis was restricted to an evaluation of the results of various simulations with different parameters for three years, for two different soil types (a sand and a light clay). The sensitivity analysis was carried out to illustrate the significance of the various parameters with respect to the results of a simulation expressed in mm or ton per hectare. It shows that with models with many variables like PRODU, a wide range of results can be obtained with relatively small changes in the various parameter values. It also emphasises that the results should be interpreted with care. However, it does not mean that the results cannot be used at all. If various variants of water management policies are compared the absolute results expressed in ton per hectare are not important as such. The results which are used to compare variants, as will be done in Section 2.3., are the percentages damage, (the relative results), and these are not much influenced by the value of a single parameter. This holds true as long as these parameters have been given reasonable values that are equal for all variants.

The data used in the sensitivity analyses

The choice to reduce the analysis to three years only which is enough for illustration purposes made it necessary to determine which years must be taken from the dataset available. A few considerations can be applied for such a selection. Depending on the interests the total rainfall or the total rainfall surplus over the whole year or over the growing season can be taken as a selection criterium or the total potential evapotranspiration over the whole year or over the growing season can be applied as such. In this case it is thought more important to look at the quatity and at the variation of the rainfall in the three most important periods, being March to May with repect to workability conditions in spring, June to August with respect to moisture availability in summer and September to October with respect to workability conditions at harvest. The philosophy behind this is that with respect to workability conditions, for example, the amount of rainfall as such might not be important but the variation of the rainfall may be the cause that access to the soil is not possible. The years were chosen from Fig. 2.17, which gives the monthly precipitation for all 30 years. The diagram shows clearly that with respect to the monthly volume of the precipitation as well as the monthly variation in the metioned periods of time, 1958 was an 'average' year, 1976 was a dry year and 1979 had a wet spring. These three years are selected to demonstrate the sensitivity of the model results for variations in the above metioned parameters.

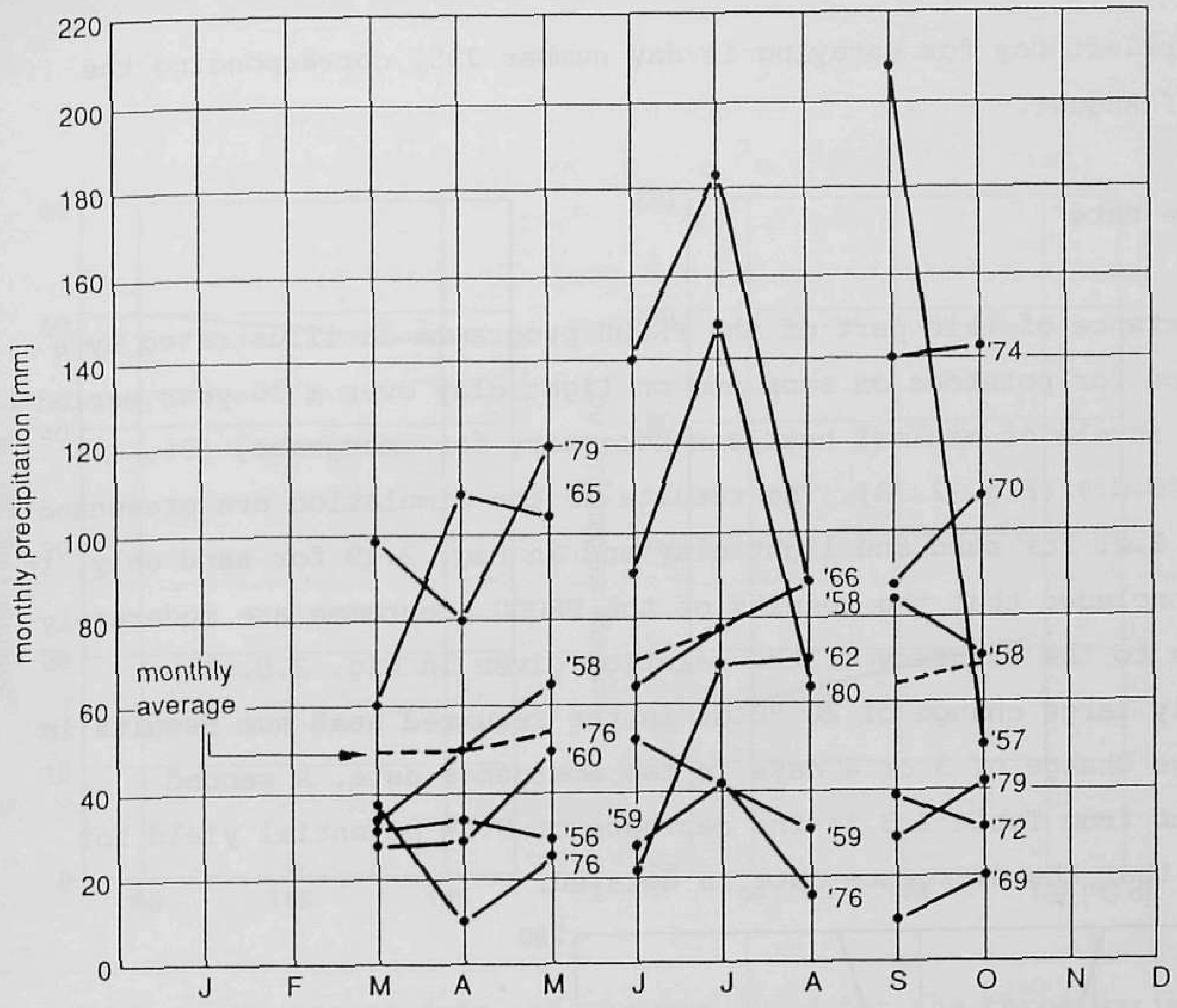


Fig. 2.17. Monthly precipitation rates over a period of 30 years, De Bilt, 1951-1980.

Other standard data for all variants are:

- the earliest day to start the preparation of the ground and planting is day number 100 which corresponds mostly to the 10th of April;
- drain depth is 1.20 m. with respect to soil surface;
- drain spacing for light clay and sand are 14.0 and 20.0 m respectively;
- the "on" and "off" levels for the pump during the summer (from day number 80, 11th of March, to day number 255, 12th of August) are -1.00 m and -1.20 m respectively, with ground level as reference level;
- during winter these levels are -1.30 m and -1.50 m respectively;

- the earliest day for spraying is day number 255, corresponding the 12th of August.

Emergence date

The importance of this part of the PRODU programme is illustrated by a simulation for potatoes on sand and on light clay over a 30-year period at three levels of minimal heat sum necessary for emergence, (96, 116 and 136 °C.d.) (Fig. 2.18). The results of the simulation are presented in Table 2.2. for sand and light clay and in Fig. 2.19 for sand only. It can be concluded that the results of the PRODU programme are moderately sensitive to the accuracy of the relation given in Fig. 2.8. A relatively large change of 20 °C.d. in the required heat sum results in an average change of 3 or 4 days in the emergence date. A second conclusion from Table 2.3 is the decrease of 0.8% potential yield for each day that the emergence date is delayed.

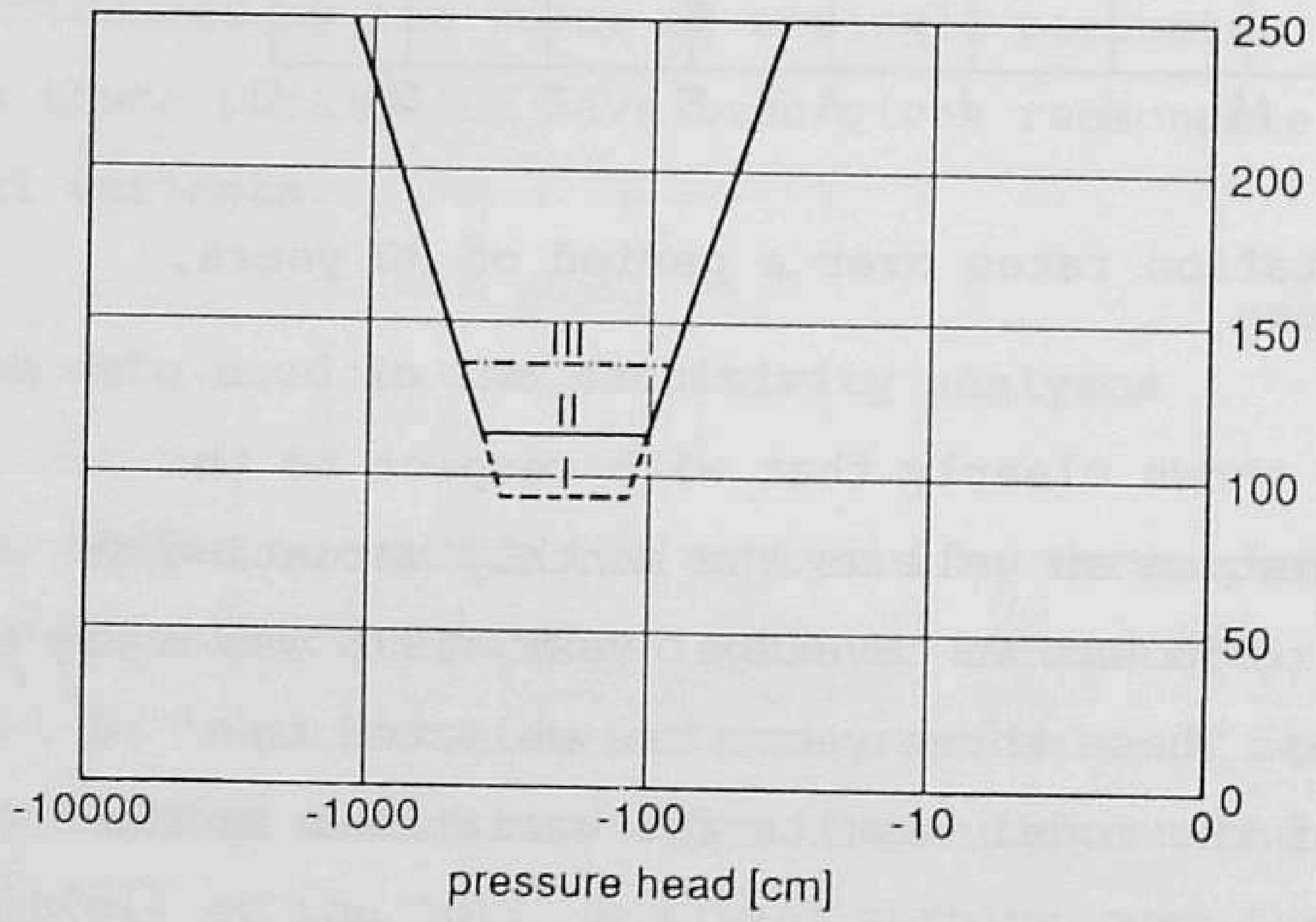


Fig. 2.18. The three variants (I, II, III) of the sensitivity analysis on the necessary heat sum for emergence.

Table 2.2. Average emergence date, the average potential and actual yield for the three variants of the required heat sum for potatoes on sand and on light clay.

variant	sand			light clay		
	average em.date [day.no]	average pot.yield [ton.ha^{-1}]	average act.yield [ton.ha^{-1}]	average em.date [day no.]	average pot.yield [ton.ha^{-1}]	average act.yield [ton.ha^{-1}]
I	123	50.2	43.5	134	45.3	39.2
II	127	48.8	42.5	137	44.4	38.5
III	130	47.2	41.3	140	43.1	37.7

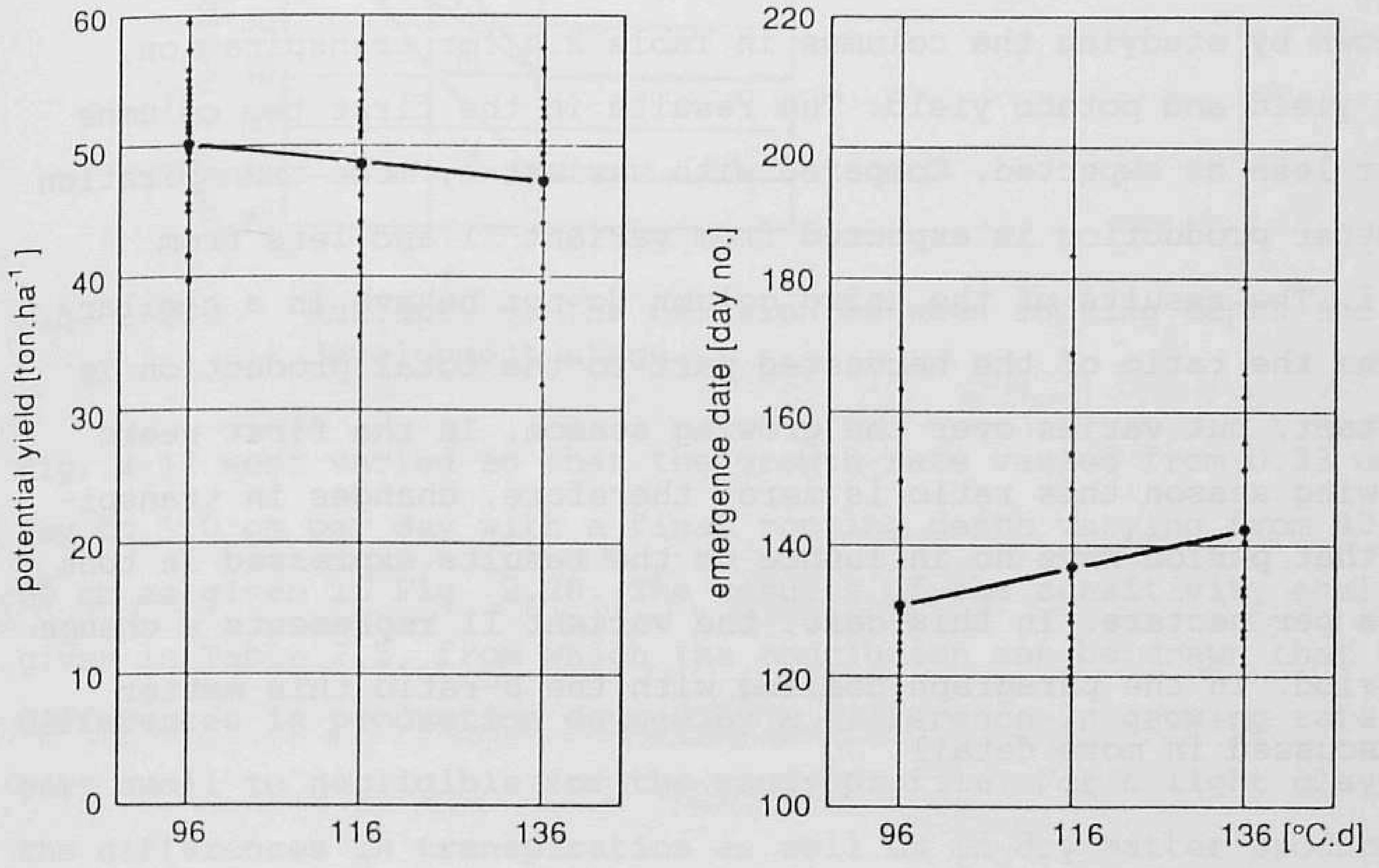


Fig. 2.19. Emergence date and maximum yield for the three variants of the minimal necessary heat sum for potatoes on sand.

Soil cover

To investigate the sensitivity of the results to the values of $D_{s,c}$ and $D_{s,m}$, these were varied within limits as given in Table 2.3. Taking variant I as the standard, variant II deviates from the standard by a period of full ground cover that is approximately 20 days longer. Variant III deviates from the standard by an earlier start of maturing by approximately the same length of time. Table 2.4 gives the results of the runs related to the sandy profile. The differences in the results

Table 2.3. The three variants of the soil cover development stage relation.

Variant	$D_{s,c}$ [-]	$D_{s,m}$ [-]
I	0.275	0.70
II	0.15	0.70
III	0.275	0.50

are considerable, which indicates the importance of the accuracy of this parameter in case the results of the simulation, expressed in tons per hectare, are important. That the results depend on many parameters is clearly shown by studying the columns in Table 2.4 for transpiration, dry matter yield and potato yield. The results in the first two columns are more or less as expected. Compared with variant I, more transpiration and dry matter production is expected from variant II and less from variant III. The results of the third column do not behave in a similar way, because the ratio of the harvested part to the total production is not a constant, but varies over the growing season. In the first weeks of the growing season this ratio is zero; therefore, changes in transpiration in that period have no influence on the results expressed in tons of potatoes per hectare. In this case, the variant II represents a change in that period. In the paragraph dealing with the β-ratio this matter will be discussed in more detail.

Table 2.4. Results of the various variants for the soil cover function for the years 1958, 1976 and 1979 relating to the sandy profile.

		total transpiration [mm]		total dry matter [ton.ha^{-1}]		total potato yield [ton.ha^{-1}]	
		pot.	act.	pot.	act.	pot.	act.
1958 emergence date: 118							
variant	I	315.9	313.8	21.4	21.3	49.1	49.0
variant	II	334.4	332.9	23.1	23.0	49.1	49.0
variant	III	299.7	297.4	19.7	19.6	42.4	42.3
1976 emergence date: 115							
variant	I	498.0	313.3	19.8	14.1	50.3	33.1
variant	II	526.9	301.7	21.5	14.4	50.6	31.0
variant	III	468.1	297.0	18.4	13.3	43.8	30.0
1979 emergence date: 127							
variant	I	303.2	300.6	18.8	18.7	44.7	44.5
variant	II	318.3	316.5	20.0	19.9	44.2	44.0
variant	III	279.7	280.7	17.4	17.4	39.1	39.1

Depth of the root zone

That the sensitivity of a crop to drought strongly depends on the rooting depth is one of the major conclusions from the sensitivity analysis that has been carried out. The values of the parameters from Fig. 2.12 were varied so that the growth rate varied from 0.33 cm per day to 1.0 cm per day with a final rooting depth varying from 40 cm to 60 cm as given in Fig. 2.20. The results of the sensitivity analysis are given in Table 2.5, from which the conclusion can be drawn that the differences in production caused by a difference in growing rate are very small to negligible for the sandy profile. For a light clay profile the differences in transpiration as well as in dry matter production and potato yield for the four variants were even smaller. These results endorse the opinion that at the beginning of the growing season, when roots are growing at a relatively low transpiration rate and a profile almost at field capacity, the required amount of water can be obtained easily enough. The pressure head profile is such that the α-values of

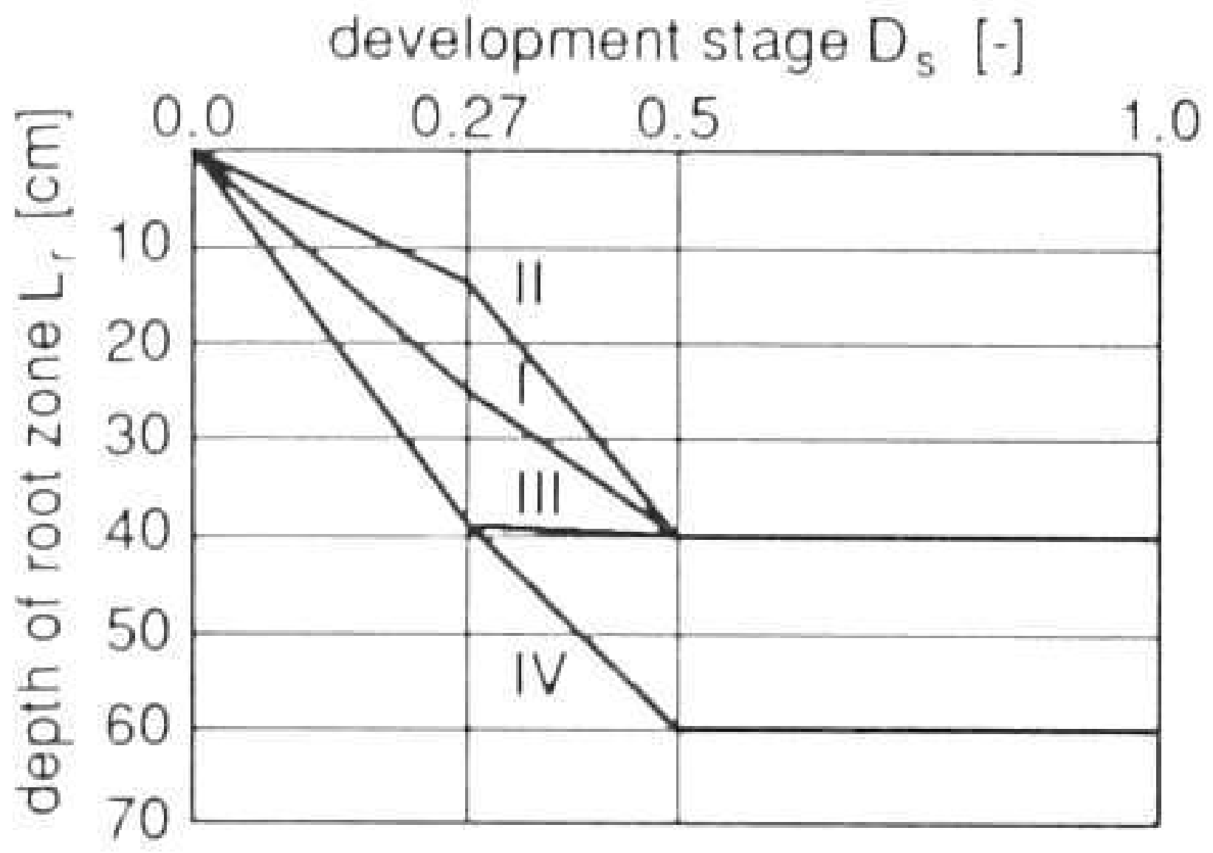


Fig. 2.20. Variants of the relation between rooting depth and development stage.

Table 2.5. Results of the four variants for the relation between rooting depth and development stage relation for the years 1958, 1978 and 1978 for the sandy profile.

		total transpiration [mm]		total dry matter [ton.ha^{-1}]		total potato yield [ton.ha^{-1}]	
		pot.	act.	pot.	act.	pot.	act.
1958	I	315.9	307.9	21.2	20.9	48.9	48.9
	II		313.5		21.2		48.9
	III		314.0		21.2		48.8
	IV		315.5		21.2		48.9
1976	I	498.4	302.6	19.7	13.7	50.1	33.2
	II		308.7		13.9		33.0
	III		306.1		13.8		31.5
	IV		346.3		14.8		35.5
1979	I	296.2	292.1	18.4	18.3	43.8	43.6
	II		294.0		18.3		43.8
	III		295.0		18.3		43.8
	IV		296.2		18.4		43.8

the sink term are at their maximum and the full capacity of the roots is not necessary to fulfil transpiration demands. Later in the growing season, when drought damage can develop, almost equal yields are obtained when the rooting depth is equal in the cases considered. If variants with different rooting depths are compared, the conclusion is justified that the differences increase according to the dryness of the year. Deeper rooting depths achieve better yields and a lower risk of drought damage.

2.4 Evaluation of results

In the following Chapter two examples of the application of PRODU are discussed. Before passing to these examples, the problem of the comparison of different water management policies is dealt with in this section. The examples are elaborated differently, while the comparison of the results is always the same and based on the following reasoning.

After 30 years of model simulation all variants j result in a potential yield Q_{pot} and an actual yield Q_{act} for each year i, in notation form: $Q^j_{pot,i}$ and $Q^j_{act,i}$ respectively. To make an overall comparison of all variants, the maximum potential yield is determined for each year covering all variants:

$$\max.\ Q^j_{pot,i} \text{ for given } i = Q^j_{pot,i,max} \qquad (2.28)$$

The relative yield in year i for variant j:

$$Q^j_{rel,i} = (Q^j_{pot,i,max} - Q^j_{act,i}) / Q^j_{pot,i,max} \qquad (2.29)$$

The yield depression in year i for variant j:

$$Q^j_{dep,i} = 1.0 - Q^j_{rel,i} \qquad (2.30)$$

This results in 30 yield depressions for every variant under consideration, which can be subdivided into 10 intervals of yield depressions, the classes 0 -10, 10 - 20 ..... up to 90 - 100 %. Presented in a histogram this leads to diagrams like that in Fig.2.21. In all the cases studied this histogram can very suitably be approximated by a lognormal distribution as is also indicated in Fig. 2.21. The underlying assumption

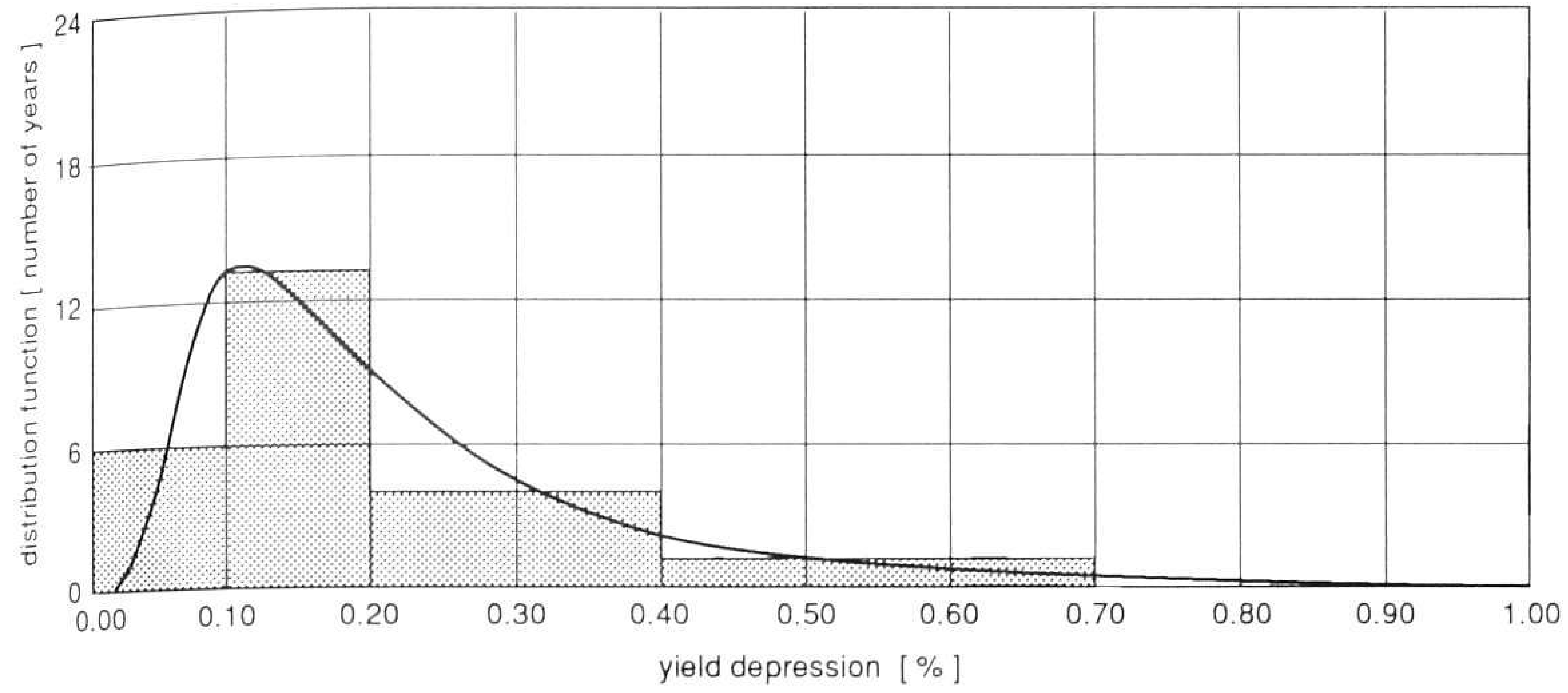


Fig.2.21. Histogram and corresponding log-normal distribution (an example).

which makes this a valid consideration is that the climatological conditions of the thirty years are presumed to be a representative sample of all climatological conditions that can occur. If this is done for all variants under consideration, the variant representing the most preferable way of water management, can be selected by comparing the probability of yield depressions of more than a particular given value.

A criterion for this comparison could be that the variant with the lowest probability of yield depressions of more than 10 % will be selected. Interesting results can be obtained by the comparison of various levels of yield depressions in this way, because in general not every comparison will result in the same variant as the most preferable. Finally it must be stressed that if in a case under study one or more variants are added to the comparison at a later stage of the study, each comparison must start with the original results of all simulations. It is not inconceivable that the added variants result in higher potential yields in some years as compared to the other variants, resulting in higher yield depressions for all earlier variants.

CHAPTER 3 APPLICATIONS OF PRODU

3.1 Introduction

The PRODU model was developed to evaluate various water management regulations by their influence on the productivity of agricultural areas. The concept of 'water management regulations' implicates all parameters which influences the water level and the water level variation. In general three main classes can be distinguished.

Firstly, those parameters which are related to the hydrological characteristics of the area and the variation of these characteristics in space and time. This compact formulation comprises a wide range of parameters: soil types and their characteristics with respect to permeability, infiltration capacity, drainage density, the shape of the area, the length and the width of the area, the land use and its effect on infiltration and drainage. To the second class belongs parameters like the open water storage area, the pump capacities for in- and outflowing quantities of water and the characteristic of the inlet and outlet weirs. The parameters in this first and second class are related to the physical features of the area which can not be changed easily. The third class of parameters can be characterized by the consideration that they can be changed relatively easily: it concerns the levels of the sensors for switching the pump on or off, the crest heights of the weirs and the variation of these levels during the year. It comprises not only the discharge of water but also the water supply possibilities.

In this chapter the applicability of the PRODU model is demonstrated by examples which concentrate on parameters of the third group in Section 3.2 concerning the choice of the date for switching from winter to summer level and on variables of the second group in Section 3.3 concerning the relation between the open water percentage and potato yield. The model and the results that can be obtained with it are discussed in Section 3.4.

3.2 Choice of the date for switching from winter to summer level

Introduction

Traditionally, good workability conditions in spring and at harvest-time in autumn in the polder areas in the Netherlands are guaranteed by a water management practice which distinguishes a summer level and a winter level of the surface water in the polder. In winter the level is in general lower than in summer. The moment to switch from the one to the other is chosen in such a way that at the appropriate moments, either the drainage conditions or the water supply conditions are optimal. With this practice of water management two decision problems can be distinguished: the choice of the date to install summer level and the choice of the date to establish winter level. This section focusses on the first problem. The second problem has also been subject to a study from which the conclusion was drawn that this switch date is of minor importance even with respect to the workability conditions at harvest. The workability conditions at the end of the growing season appeared to be mainly determined by the drain depth, the drain distance, the functioning of the sub-surface drainage system and the existence of a plough sole.

The first problem, the choice of the date to establish summer level, presents more interesting aspects. The number and the importance of these aspects vary with the possibilities of the Waterboard to control the levels. The choice of the date to establish summer level, that is, as such, merely concerned with the problems relating to the workability conditions, can implicitly determine the total available amount of water in the soil for the growing season. In the polders on the islands in the south west region of the Netherlands, there are no possibilities of supplying water of reasonable quality during the growing season because these islands are completely surrounded by salt water. The higher summer level can only be realized by storing the rain which falls after the establishment of the summer level. The decision problem in these circumstances can be characterized by the following reasoning: the earlier the establishment, the more rain can be stored, the less the chance of drought damage, but the greater the chances of poor workability conditions. In other polder areas in the Netherlands there are better supply possibilities in summer but the authorities are confronted with water of a quality that can have negative effects from an ecological point of view. It is for that reason that in these cases as well the water management regime aims at water conservation.

In this section the above problem is considered, first for the case with no supply possibilities, later extended with possibilities for thw suply of water. It is discussed for two different soil profiles, a light clay and a sand profile. The part of this section relating to the case with no supply possibilities can be regarded as a brief enumeration of results reported by Volp and Plywaczyk (1987).

The Model Polder

The model polder has a total area of 100 hectares of which 3 % consists of paved area and 3 % of open water surface. This is typically an agricultural polder where the paved area consists of roads, farmyards and the roofs of those buildings that do not discharge the rainfall water into the sewersystem. The area of open water is presumed to be independent of the actual water level. This is a justified assumption as long as water levels remain within reasonable limits. The pump capacity is 13 mm/day, with "on" and "off" levels of -1.30 m. and -1.50 m. in winter and -1.00 m. and -1.20 m. in summer respectively. These levels are expressed in relation to soil surface. The soil profiles are:

- A homogeneous profile of light clay of a depth of 4.00 m., with a cultivated layer of 0.40 m. on a layer of poor permeability. The sub-surface drainage is designed according to the usual design procedures, resulting in a drain depth of 1.20 m., a drain spacing of 14 m. and a drainage resistance of 20 days. The open drains into which the sub-surface drains discharge are placed at 300 m. intervals.
- A homogeneous profile of fine sand, again with a thickness of 4.00 m. on a poorly permeable layer. The choice of a drain depth of 1.20 m. results in a drain spacing of 20 m. and a drainage resistance of 18 days. The spacing of the open drains remains 300 m.

For both profiles four variants are considered, which differ only in the date of switching from winter to summer level, i.e., the day numbers 80, 100, 120 and 150. For the clay profile a variant is added in order to obtain an optimum, i.e., day number 180. These day numbers correspond respectively to 21 March, 10 April, 30 April, 30 May and 29 June.

On evaluating the results of the thirty years of simulations for each variant, the values of the potential and the actual yield expressed in tons of potatoes per hectare are found to vary, as do the values of the date of the end of soil preparation, the emergence date and the mean values of groundwater levels. The variation of the average values of the ground water level over the year, with respect to soil surface is given in Figure 3.1, in combination with the average values of actual potato yield.

With reference to these results the following comments can be made. Generally speaking, a later switch date tends to result in an earlier emergence date and thus in higher potential yields, resulting in higher actual yields for the clay profile because of the good capillary properties of this soil. This does not hold true for the sand profile. The emergence date is not moved further forward if the last three variants are considered, while the actual yield is reduced by the poor capillary properties of the profile. The profits of an earlier emergence date are reduced by drought damage later in the growing season.

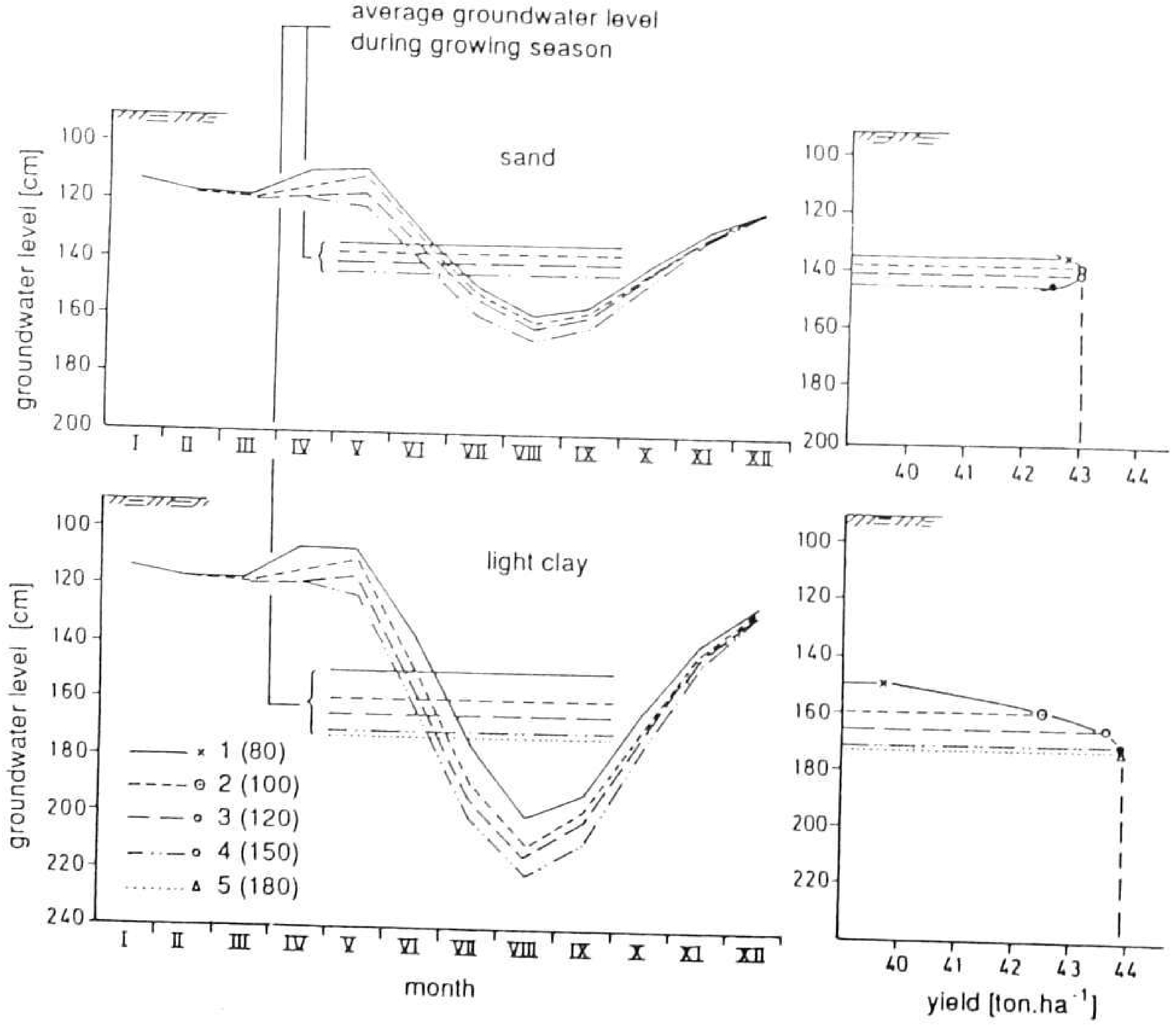


Fig.3.1. The thirty year average values (1951-1980) of groundwater levels and actual potato yields for the two soil profiles and the various variants.

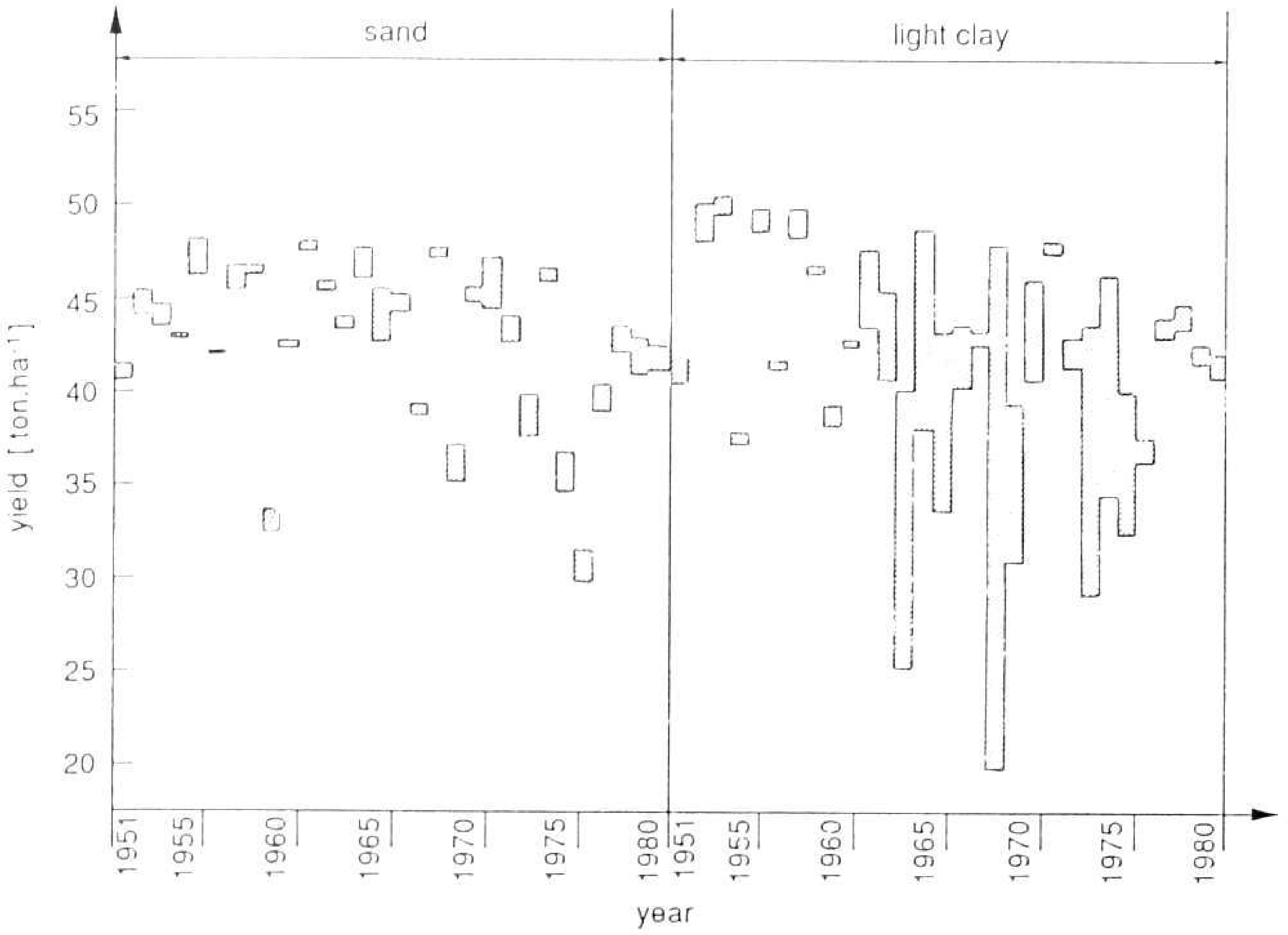


Fig.3.2. The yearly variation of the actual potato yield for all years in the period 1951 - 1980 for the two soil types.

These are statements based on the consideration of the thirty year average values as given in Fig.3.1. In this figure the thirty year average values (1951-1980) of groundwater levels and actual potato yields for the two soil profiles are given for the various variants. That the results greatly depend on the meteorological conditions in a particular year is illustrated in Fig. 3.2, where the variation of the resulting actual yield are given for the two soil types for each year in the period 1951 - 1980, if the date of switching from winter to summer level is varied. Notably the results for the clay profile vary considerably in several years. This is due to the variation in emergence date, which is mainly determined by the workability conditions in spring. In years with a dry or fairly dry spring, as occurred during the period from 1950 to 1959, the results show much less variation than they did the years that followed, which had rather wet springs. If the two soil types are compared the smaller variations for sand are notable. This can be explained by considering that sand, in comparison with clay, has better bearing properties at higher moisture contents and poorer capillary properties, so that it dries more easily to workability limits.

The results of the four variants of the sand profile and the five variants of the clay profile have been statistically analysed, as discussed in the introduction to this section. On considering the results of the exercise, presented in Fig. 3.3., it is clear that the chances of damage, wet and drought combined, are reduced by postponing the switch date. This is particularly true for the clay profile, where, because of its strong capillary properties, drought damage plays a role of minor importance. The results for the sand profile are indifferent in the period from day number 100 to day number 120, after which the chances of damage increase because of the poorer capillary properties of the profile. The different properties of the two soil types are expressed clearly by the existence of a minimum for the sand profile in the interval around day number 100 in contrast to the course of the curves for the light clay profile where apparently an optimum is obtained after day number 150.

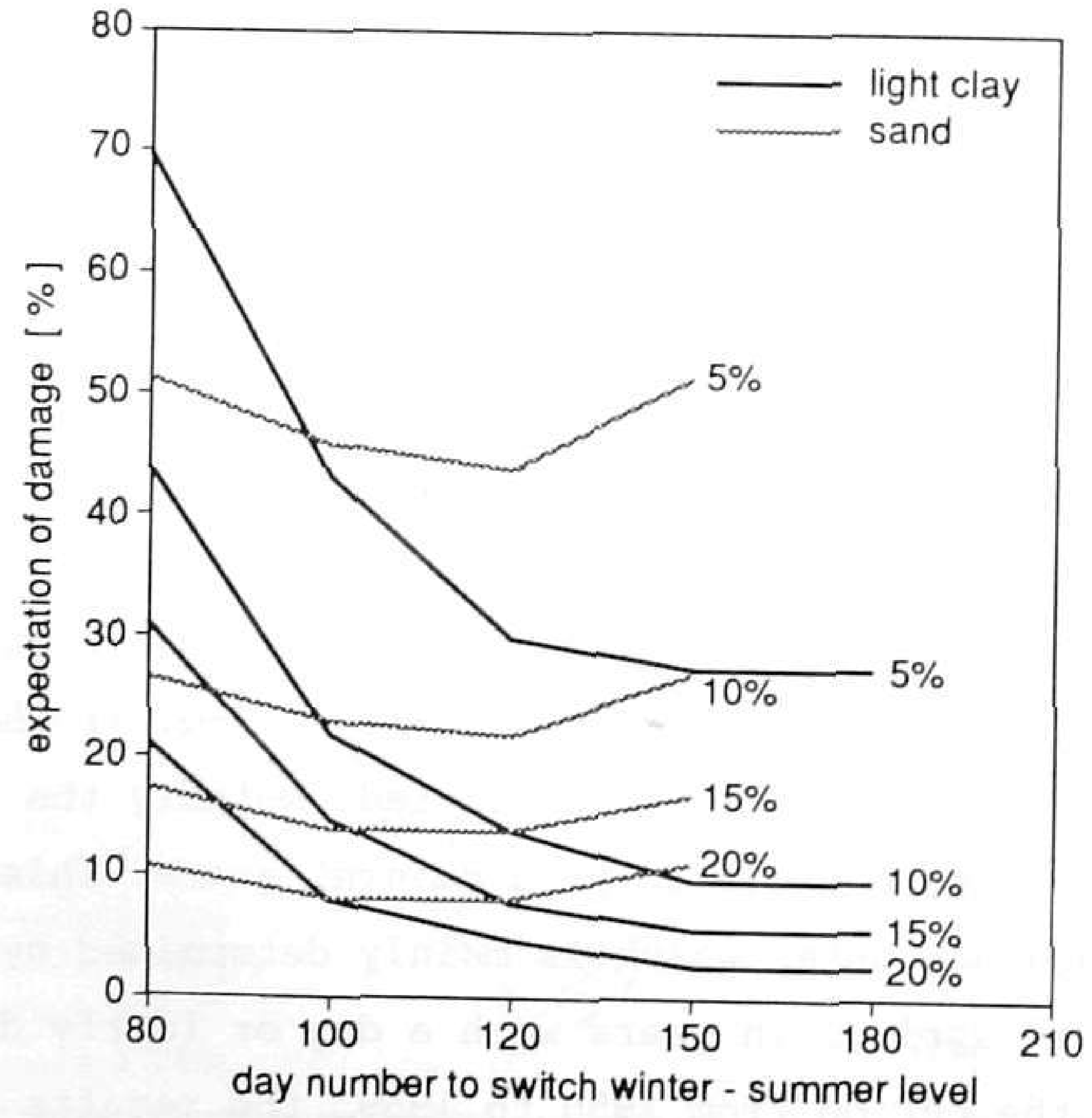


Fig.3.3. The chances of yield losses of a stated magnitude in relation to the date of switching from winter to summer level for the clay and the sand profile.

In order to evaluate whether more sophisticated water management can yield better results the following variants have been added for consideration:

- variant I is aimed at the minimization of the period up to planting date; summer level is established only after all necessary workable days have been realized (variant I);
- in order to prevent a part of the possible drought damage that may be caused by a late switch date according to variant I, in variant II a level is chosen halfway between winter level and summer level in the period between the switch date realized and day number 80 (for the clay profile);
- variant III is equal to variant I but with a water supply capacity of 2 mm per day which is available from the planting date.

In the following only the results for the clay profile are discussed, as the results for the sand profile are similar but far less pronounced. The results of this extension for the clay profile are given in Fig. 3.4. For comparison purposes the results of the earlier variants are also taken into consideration and presented in Fig. 3.4.

By adding the results of the simulations of the variants I, II and III to the statistical analysis it was found that in some years better

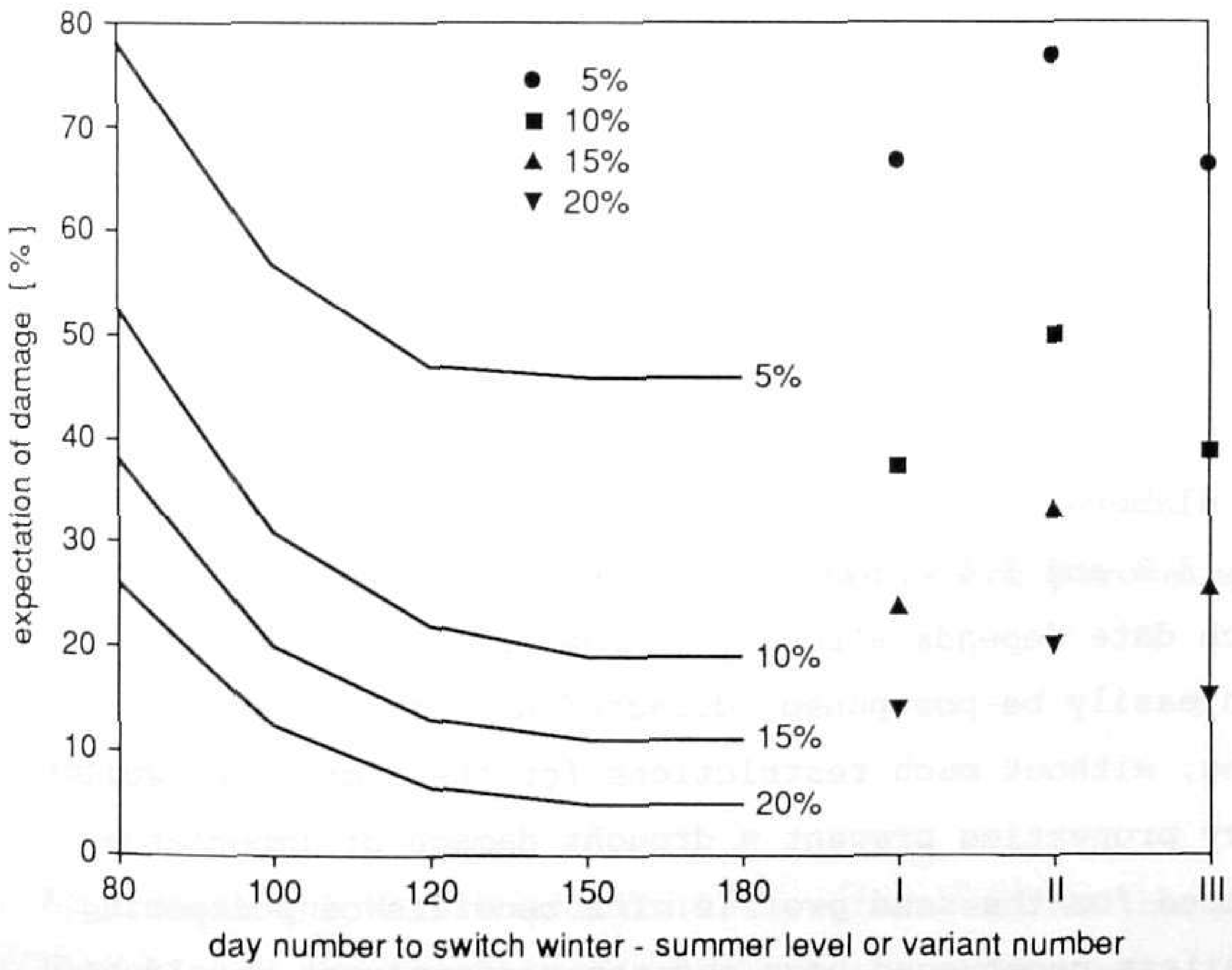


Fig.3.4. The chances of yield losses of a particular value in relation to the various dates to switch from winter to summer level and the three other variants considered for the clay profile.

potential yields were obtained in these new variants. This manifested itself in better average values but more clearly in higher chances of yield damage in the earlier variants. From the comparison of the results presented in Fig. 3.3. and Fig. 3.4, for example, it can be seen that the chance on a yield loss of 5% and 10% of the variant with a fixed switch date of the 10th of April (day number 100) increased from 44% to 57% and from 25% to 31%. These higher potential yields, however, correspond to equal or lower actual yields in variants I, II and III, resulting in higher chances of yield damage, as compared with the earlier variants. This can be explained by realizing that with the water management policy of the variants I, II and III the problems of workability in spring are adequately solved, but that it disregards the problems of adequate moisture availability during the summer. The higher chances of yield damage in variant II as compared with variant I, are accounted for by delayed emergence dates.

The effects of the water supply in variant III, aiming at maintaining the summer level and thus supplying water to the plant by infiltration, on actual yields are negligible as compared with variant I. That the water supply is not very effective is caused by the fact that the available head for infiltration is only 10 cm. on average. The supply capacity of 2 mm is enough to maintain summer level.

It is assumed that the positive effects of the water supply can be increased if the water is added directly to the root zone by sprinkler installations. This was not subjected to further research.

Conclusions

The results of the elaboration of the subject of this section are summarized in Figs. 3.3 and 3.4. Figure 3.3 makes clear that the optimal choice of the switch date depends strongly on the soil characteristics. The switch date can easily be postponed, directed at better workability conditions in spring, without much restrictions for the light clay soil. Its strong capillary properties prevent a drought damage of importance. This can not be stated for the sand profile. The benefits of postponing the switch date are less pronounced here and the postponement should be handled with care for the resulting drought damage during the growing season may be larger than the diminished 'wet' damage caused by its poorer capillary properties.

The results presented in Fig.3.4. make clear that a more sophisticated way of water management with respect to levels, switch data, as well as - in combination with the results of van Wijk and Feddes (1986) -, drain depths and drain spacings, all of this directed at better workability conditions in spring, does not always result in higher yields. If the results of the 'more sophisticated' water management policies (variants I, II and III) are compared with the results for a policy of a fixed a priori chosen date, it can be stated that 'more sophisticated' aimed at workability conditions in spring may be justified only if it is combined with a more sophisticated water management procedure with respect to water supply, i.e., sprinkler installations. Depending on the soil characteristics, there is a chance that the better yields of the policy aimed at better workability conditions in spring only, remain potential yields, combined with higher chances of yield damage or, more specifically, drought damage.

If water management is aimed at the combination of workability conditions in spring and at the water supply during the growing season, there are two important advantages associated with this water management procedure. It results in higher yields and it gives a lower chance of yield damage, resulting in a more stable income for the farmer. This conclusion concerns the above given weighing of variants and does not include the cost aspects of the various water management policies. The question whether the higher and more stable income values the investment and exploitation cost of such a sprinkler installation is not aswered here.

3.3 Relation between the open water percentage and potato yield.

Introduction

At present the Waterboards in the western and northern part of the Netherlands are to an increasing extent confronted with demands by farmers for permission to fill up ditches. The results of a limited inquiry into this matter indicate that approximately 25 % of the demands for permits issued by Waterboards are related to this subject (Volp and Asperen, 1987). The advantages to the farmers are:

- greater possibilities for more economical work organization on the plot, because of more optimal plot sizes for larger agricultural equipment;
- increase in total productive area;
- reduced length of channel slopes to maintain; in these cases it concerns tertiary ditches with a drainage function for the water originating from the primary sub surface drainage; a collector drain will be necessary to replace the function of the ditches.

Filling up these ditches entails some disadvantages:

- The reduced area of open water and thus the storage capacity, generally speaking, results in higher water levels in these polders under drainage conditions. This will cause more drainage water to be stored in the soil profile, with possible effects for agriculture. In spring and in autumn it can be important for the workability conditions, resulting in delayed emergence dates and a possible loss of part of the yield at harvest. During the growing season it may have positive effects with respect to decreased drought damage.
- Inspection and maintenance of the primary sub surface drainage will require more effort.
- The costs of the filling and the construction of the collector drain represent an aspect that is not unimportant.

In arriving at the decision for the permission other aspects should be given due weight. Apart from the agricultural and water management aspects, legal and economic aspects can be important. These have been dealt with in more detail by Volp and Asperen (1987). They found the total of the cost of the filling material and the levelling to be crucial to the cost-benefit ratio. Given the cost of levelling, not much could be reserved for the acquisition of the fill material. This cost-benefit analysis did not include the benefits originating from more economical work organization possibilities, such as lower exploitation costs for machinery, economizing of working hours and a lower level of nuisance. Including these aspects will justify the spending of more money for the acquisition of the fill material. This section will focus on the water management aspects of the above problems, in order to find an approach for decision making to grant or to reject the application for permission to fill the ditches. In other words, an attempt to determine a lowest permissible percentage of open water surface.

General information

The data for this case were adopted from a research done for the Waterboard "De Groote Waard" situated in the south-western part of the Netherlands. For detailed information about the extensive data collection reference is made to Asperen (1986); for the schematization into 12 variants with detailed results to Asperen and Volp (1986). The results of this study will be discussed with reference to the results of two of the twelve variants. For all variants the following data are valid.

The soil profile consists of a homogeneous profile of light loamy soil, and a cultivated layer of light clay with a thickness of 0.30 cm. The sub-surface drainage is located at a depth of 1.30 m. below soil surface, the drain spacing being 12.0 m. The open drains into which the sub-surface drains discharge, spaced at 300 m, are assumed to be replaced by collector drains if they are filled up. A pump with a capacity of 12 mm per day is operated in such a way that winter and summer levels can be distinguished. Summer level is installed from April 15th (day number 105) to October 16th (day number 290). There is a water supply possibility from 30 April to 2 September with a pump capacity of 12 mm per day, an "on" level of 10 cm. below the summer "off" level and an "off" level equal to the summer "off" level. The variants have a paved area covering 3 % of the total area, which discharge the rainfall directly into the open water. The variants have different summer and winter "on" and "off" levels as given in Table 3.1. Each variant can be subdivided into three sub variants a, b and c with an open water percentage of 0.5, 1.5 and 3.0 % respectively.

Table 3.1. The summer and winter "on" and "off" levels for the pumping station of the two variants with respect to soil surface.

Variant	Summer		Winter	
	m with respect to soil surface			
	on	off	on	off
I	1.50	1.60	1.80	1.90
II	1.40	1.50	1.60	1.70

Results

The differences in the results of the two variants, and for each variant the three sub variants, expressed in a potential and an actual yield in ton per hectare and the emergence date were found to be negligible. In order to gain some insight into the influence of the percentage open water, the results of the two variants have been subjected to the same statistical analysis, as discussed in section 2.4. The results for the two variants are given in Fig. 3.5. With regard to these results it must be stressed at first that the differences are small. One aspect of the results, however is somewhat unexpected in some respects and needs an explanation.

The most noticeable result is the decrease of the chance on damage of all classes of damage in both variants in the comparison of the 1.5 and the 3.0 % sub-variants. This result corresponds to what is expected: a larger storage capacity reduces the chances on damage. This result supports the conclusion that larger storage capacities reduce the effects of extreme situations with respect to crop yield.

When interpreting this result it should be realised that the concept 'a lower chance of damage of a certain class' means a lower chance of damage due to too wet conditions on the one hand but it can also indicate a lower chance of drought damage. The justification of this interpretation of the curves is confirmed by considering the differences in the curves for the two variants. The curves make clear that the decrease is stronger in the variant with a smaller freeboard. A smaller freeboard means a higher risk of damage due to too wet conditions. In other words: this water management policy is more sensitive to extreme situations.

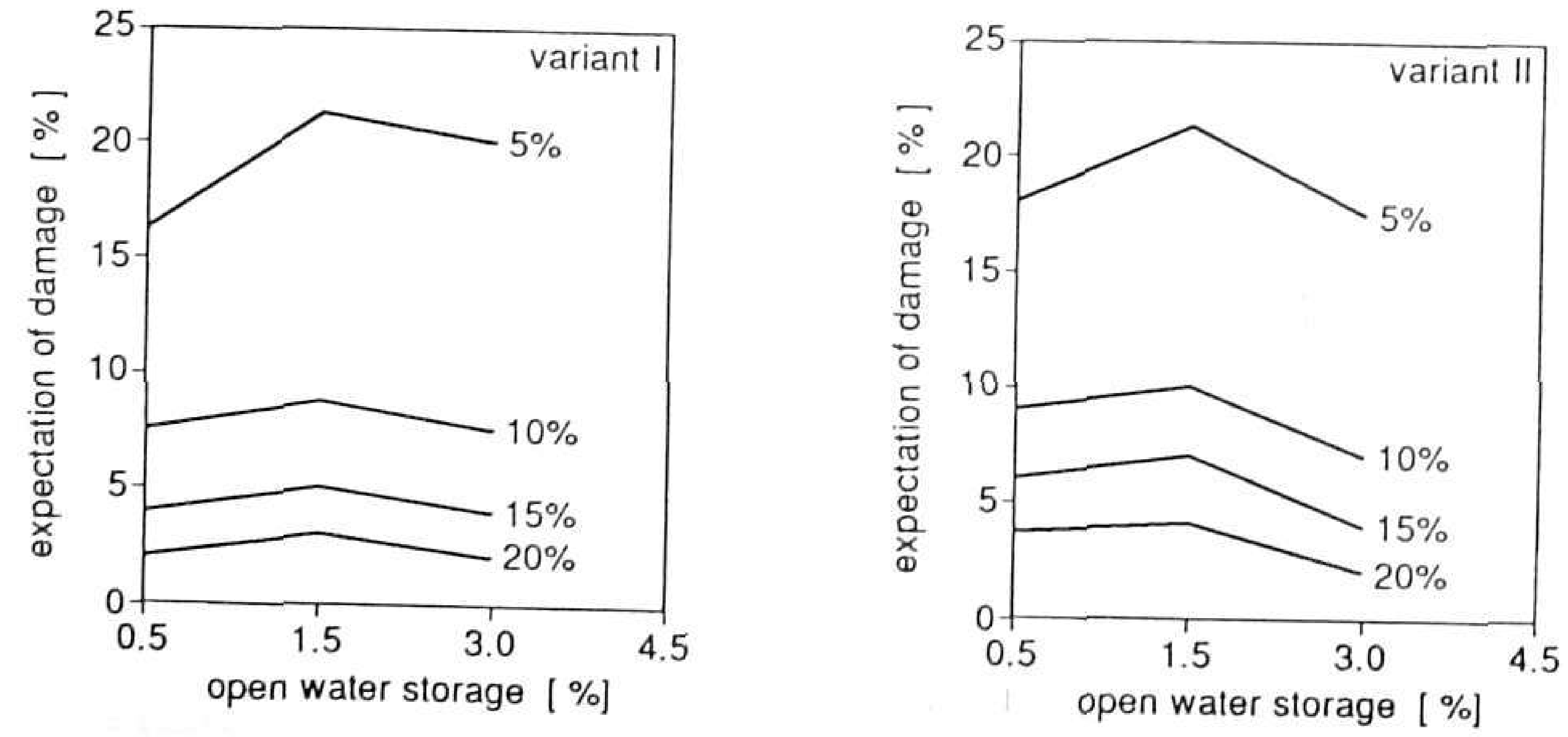


Fig. 3.5. The chances of yield losses of a particular value in relation to the percentage open water for the two variants I and II.

The above interpretation of the results seems to be disturbed by the results for the variant with 0.5% of open water surface. These results can be described as a decrease of the chance on damage of 5% (or smaller) if the open water storage is decreased from 1.5 to 0.5%, which seems to be in contradiction with the explanation of the results in the preceding passage. The interpretation of this result is complicated by the conception of damage as it is used here: as a sum of "drought" and "wet" damage. It must be stated here that this decrease holds especially for this class of damage and not (or in any case is far less pronounced) for the other classes of damage. This result can only be explained by assuming that the decrease in this class of damage of 5% (or smaller) is determined mainly by drought damage. The following reasoning is given as an explanation.

The chance of this class of damage diminishes for a diminishing storage capacity of the open water surface. A smaller open water surface results in higher levels of the open water if the pump capacity remains the same. These higher levels can have a positive effect on moisture content in the root zone during the growing season if drainage is temporarily impeded by these levels.

Apart from the above reasoning it must be stressed that the differences are small. If decisions are taken based on the curve of the 10 % damage it can be stated that from the agricultural point of view, represented by the behaviour of the average values of potato yield and by the course of the chance of yield damage, there are no serious disadvantages to decreasing the open water surface by filling in the ditches. This statement holds true as long as the 'on' and 'off'levels of the pump are not less than drain depth, the pump capacity is not less than 12 mm per day, the percentage of paved area is not significantly more than 3 %, and above all, as long as the water can be transported effectively to the pumping station. This last constraint results in the practical minimum percentage of open water of about 0.5 % for main and secondary channels.

An interesting aspect of the results has not yet been elaborated. The results indicate that there is no relation or only a very weak one between the open water storage and potato yield. This can be explained by realizing that the periods during which the reference levels are exceeded are short thanks to the pump capacity of 12 mm/day. Smaller pump capacities will result in longer such periods, which are supposed to affect the agricultural production more severely than in this case.

A similar case as variant I has been worked out with the data of variant I with a pump capacity of 8 mm.d^{-1}, here referred to as variant III. The results are given in Fig.3.6. where the results of variant I are also given for comparison purposes.

From the comparison can be seen that the variation of the chances of damage which could be distinguished between the sub-variants of variant I are subdued and cannot be distinguished in the sub-variants of variant III. It can further be concluded that the chances of yield damages of all classes are unmistakably increased: 10, 8, 7 and 6% increase for the 5, 10, 15 and 20% damage classes for example.

This leads to the conclusion that the larger exceedances with longer duration, caused by the smaller pump capacity, result in higher chances of yield damage of all classes of damage.

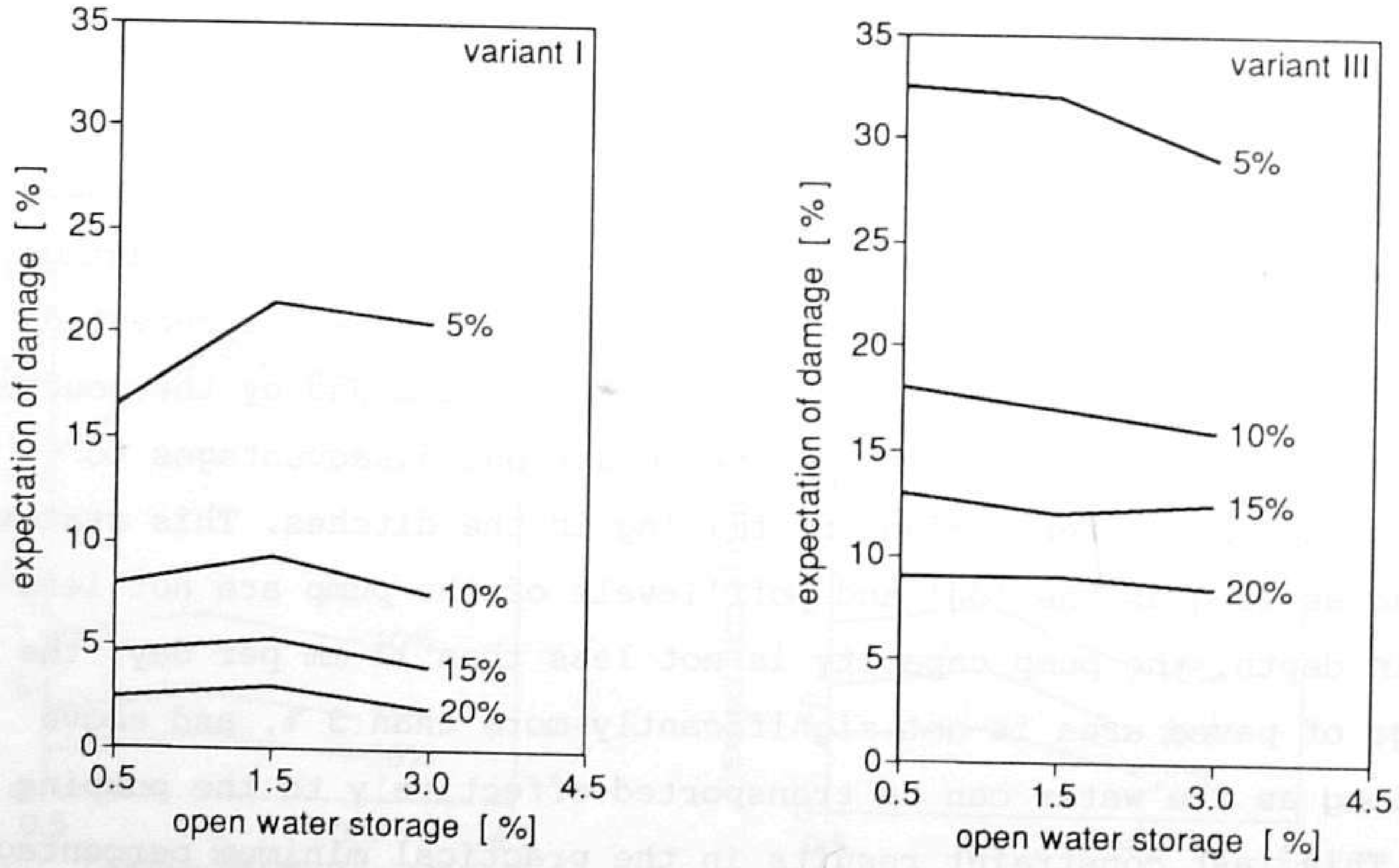


Fig. 3.6. The chances of yield losses of a particular value in relation to the percentage open water for the two variants I and III.

It is expected that these higher chances of damage are caused by the yield losses related to poor workability conditions more than to a decreased functioning of the roots, represented by lower values of the α-function for higher moisture content values in the root zone. To verify this, the same case of variant III is repeated with a higher LPL-value. That high that access to the plot can take place at any time (variant IV).

It appears from Fig.3.7 that in this variant IV the chances on a yield damage of 5% decreases to about 3%, while the chances on higher damage is reduced to zero. This important decrease demonstrates that an important part of the yield damage corresponds with the 'wet' damage and the workability limit in particular.

This example demonstrates the importance of the various variables in the design of polder systems, such as pump capacity, reference level and open water storage. It emphasizes the importance of an optimization with respect to workability conditions. In Section 3.2. it is demonstrated that an optimization should combine the 'drought' damage with the 'wet' damage. The results of the variants I, II and III in this section illustrate that exceedances of the reference level do not have to result in negative effects on crop yield and are strongly related to the degree and the duration of the exceedance.

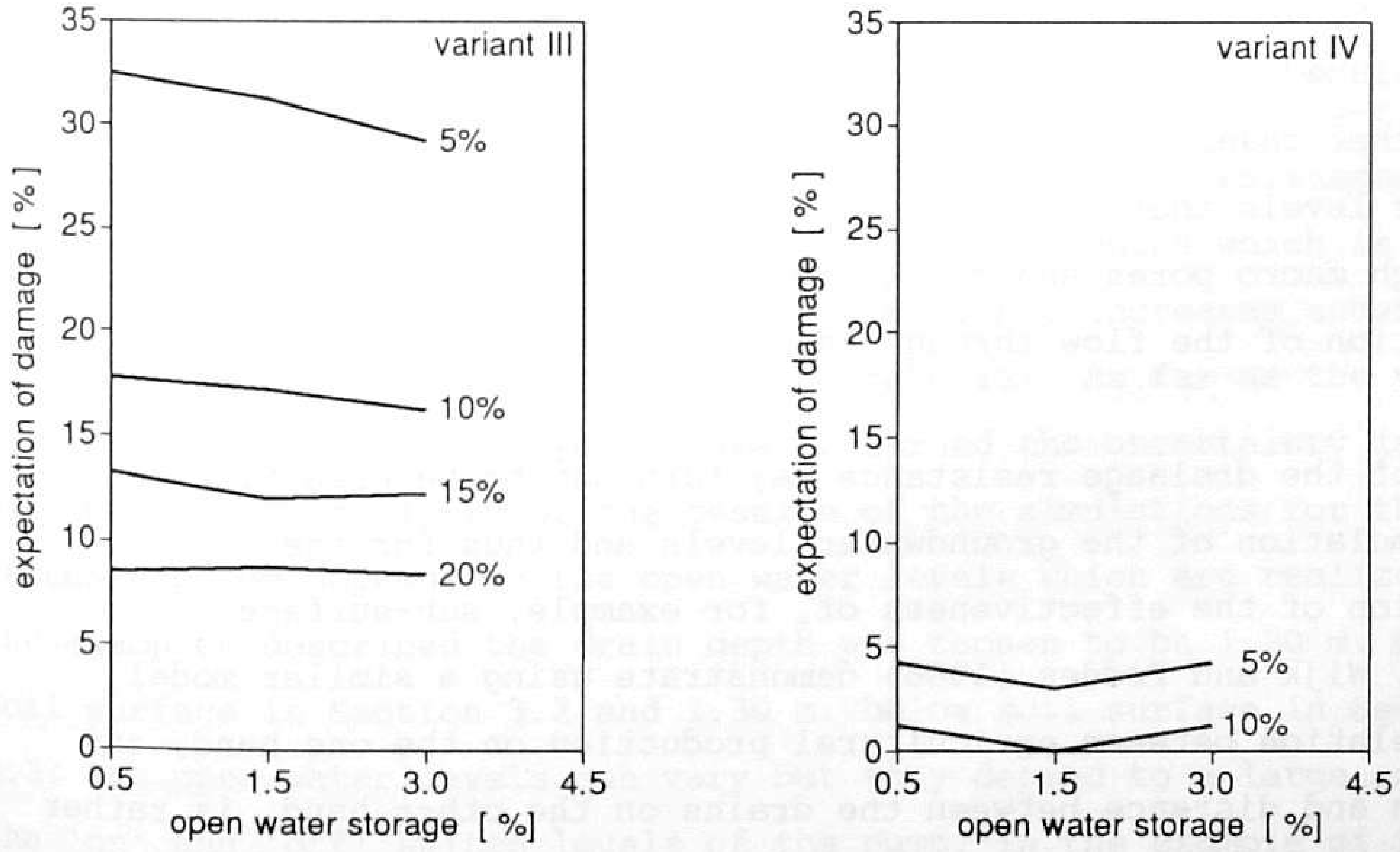


Fig. 3.7. The chances of yield losses of a particular value in relation to the percentage open water for the two variants III and IV.

3.4 Discussion

This section is dedicated to reflections about the value of the PRODU model and of the results that can be obtained with the model. Considering a computer model like PRODU the first thing meets the eye is the fact that it has many parameters. The value of such a model depends strongly on the confidence level applying to the determination of parameter values and/or on the sensitivity of the model results to uncertainties in the parameter values. The parameters can be divided into three groups:

- soil parameters; in this group parameters related to the description of the water flow through the soil profile are to be found as well as parameters related to workability.
- water management parameters; this is the smallest of the three groups; it merely contains the parameters related to the determination of the open water level;
- plant parameters; this third group contains parameters related to emergence date, crop development and crop yield.

Soil parameters

With respect to the first group, the soil parameters, it can be stated that the accuracy of the simulation of, for example, groundwater levels in a particular case strongly depends on the situation. In some cases the soil qualities like the pF - curves and the thickness of the layers that can be distinguished, will vary considerably even within the area of one plot. In other cases it may be important for correct simulations of groundwater levels that hysteresis should be incorporated and/or that flow through macro pores and cracks be simulated separatly apart from the simulation of the flow through the unsaturated zone.

The value of the drainage resistance may turn out to be essential for correct simulation of the groundwater levels and thus for the determination of the effectiveness of, for example, sub-surface irrigation. Wijk and Feddes (1986) demonstrate using a similar model that the relation between agricultural production on the one hand, and drain depth and distance between the drains on the other hand, is rather strong. This can be explained by realizing that the relation between agricultural production and moisture content in the root zone is very strong; this remains so if the groundwater level is taken in stead of the moisture content in the root zone, because the latter is strongly related to the ground waterlevel. The relation of the open water level with the moisture content in the root zone is much more indirect and is highly influenced by meteorological conditions and by the relation between the ground water level and the open water level.

Furthermore the results are sensitive to the values of the drainage resistance adopted in the calculations. The drainage resistance in equation 2.6 is equal to the sum of the radial resistance and the resistance for horizontal flow (eq.2.7). The resistance for vertical flow is neglected here for those cases where the permeability is not extremely low. The radial resistance can be determined according to equation 2.8 for homogeneous soils and small variations of the phreatic surface. On top of these three components of resistance for flow, a fourth component, the entrance resistance, can be distinguished. The magnitude of the latter component is subject to considerable uncertainty. Field experiments indicate a dependence on the direction of flow. It is assumed that due to deposition of fine material and debris on the bottom of the watercourse, the resistance for flow from the channel into the profile is higher than the resistance to flow from the profile into the channel. Based on experimental data concerning watercourses van Bakel (1986) reports that the entrance resistance, of the order of 5 to 15 % of the total resistance, is not significantly influenced by cleaning. The resistances under sub-irrigation are approximately 10% lower than those under drainage conditions which is explained by the larger wetted perimeter. The author further concludes that the total resistance did not increase during an irrigation period of five months which is inconsistent with to the idea that the resistance increases under these conditions due to sedimentation of fine material. As far as the values for sub-surface drainage systems are concerned the uncertainty is even greater. The sensitivity of the results of the simulations for the values applied depends on the open water levels which are realized. In the examples described the drain depth was chosen to be 1.20 m. below soil surface in Section 3.2 and 1.30 m. below soil surface in Section 3.3. The open water levels can vary but they depend to a large extent on the 'on' and 'off' switch levels of the pump. In the example of Section 3.2 the levels are -1.30 m. and -1.50 m. in winter and -1.00 m. and -1.20 in summer respectively. In the example of Section 3.2 these levels are below drain depth in winter. In summer the open water levels may rise above drain depth but this only occurs in the variants where water is supplied from an external source. Considering these data it is clear that the drainage situation will be determined by the sub-surface drainage system while the sub-irrigation situation will be determined by the system of open watercourses, with the exception of the two variants where water is supplied during the growing season. In that case the sub-surface drainage system is important for the effectiveness of the water supply. As only imaginary situations are considered in this case, the drainage resistance has been given a value independent of the direction of the flow.

This enumeration of factors important for the correct description of the groundwater flow illustrates the points of attention of the research in this field. This pertains especially to research in the fields of the variability of soil characteristics and of the flow conditions within and through the macro pores. The sensitivity of the model results for the workability criteria as well as the difficulties of the determination of this parameter have been discussed in Section 2.3.

Water management parameters

The water management parameters generally give lesser problems than the rest. In the routine of the determination of the open water level, two suppositions are of importance: head differences within the area under consideration are insignificant and the reservoir approach is correct. These suppositions will generally hold for polders or polder sections with the same target level. It will not hold for sloping areas where the dimensions of the watercourses are based on the greater slope of the area, resulting in smaller cross sections, smaller water depths and higher flow velocities. The higher parts of the channel will eventually run dry. PRODU should not be applied in those cases.

Plant parameters

The sensitivity of the model results to the plant parameters has been extensively discussed in Section 2.3. It is apparently not too difficult to keep the differences between measured and calculated values small, as far as the growth rate or the total (dry matter) yield are concerned.

The approach followed here concerning the determination of development stage and crop development, for example, may be subject to criticism because of its simplicity. It can be followed here because the model is applicable to potatoes only and because normal cultivation practice is to keep the crop green as long as possible and to control the duration of the growing season by spraying. Natural dying off of the crop hardly occurs. In the same section it became clear that the model is not very sensitive to a number of other parameters. This is almost certainly correct as far as it concerns the variation of parameter values within the range of reasonable limits. These plant parameters may be considered as weighting factors that determine the importance of the meteorological conditions for the crop yield.

Summarizing the above it may be stated that in a particular case the model can give satisfactory results, but the outcome of a simulation expressed in tons per hectare should be handled with care, unless the various parameter values have been derived from measured data. However the value of PRODU should not be judged by the results of an application where the model results in terms of tons per hectare are important. The value of such a model becomes obvious, however, if applied for cases similar to the examples given in the Sections 3.2. and 3.3. The model is especially suitable to compare various water management policies. In this application the importance lies in the results in terms of relative differences caused by different water management policies. Notwithstanding the complicated and non-linear relation between the plant parameters and crop yield, the results of this application are valuable. This holds true as long as plant parameter values are not changed for the different variants and have been given reasonable values, and as long as the water management variants in the comparison do not describe extreme situations with for example periods of inundation.

This latter condition has to be mentioned because of some underlying qualities of the model. The influence of the (wet and drought) damage, on the model results in terms of tons per hectare, for example is determined largely by the value of the β-ratio in that particular period of the growing season. The damage by reduced transpiration through too wet or too dry conditions at the beginning of the growing season does not result in lower potato yield in tons per hectare is concerned because the β-ratio is zero in that period.

Another point of discussion can be the schematization of the water uptake by the root system of the plant using a Sink term with an α factor. This schematization attributes to the root system a great recovering potential after periods of too wet or too dry conditions in the root zone. Too little is known about plant behaviour and especially rooting qualities to enable improvement of this schematization but there are grounds supporting some pessimism on the subject of recovering capacity.

With the restrictions and limitations given above the PRODU model remains a valuable instrument to evaluate various water management policies. Not too much value should be attached to differences of the expectation of damage if they are small, (in the order of some percents), although the explanation of the tendency in the results may be of interest. Such a result is obtained in the example of Section 3.3. where the influence of the percentage open water on crop yield is discussed. If the differences of the expectation of damage are much larger, as in the example of the choice of the date for switching from winter to summer level they do support the conclusions of that section.

CHAPTER 4 THE DESIGN OF WATER MANAGEMENT SYSTEMS

4.1 Introduction

On the subject of the design and dimensioning of systems of watercourses a vast quantity of literature is available. A distinction can be made between literature aimed at the design of irrigation systems and literature aimed at the design of drainage systems. The distinction is merely based on the difference in (complementary) design criteria and the set-up of the system. In this context one should think of sedimentation and erosion criteria, intake structures and the determination of the design discharge. Internationally well known standard books are Chow (1959), Henderson (1966), Smedema and Rycroft (1983), ILRI (1972-1974), Kinori (1970), Kinori and Mevorach (1986) and Withers and Vipond (1974). Framji et al. (1984) give an overview of the design practices for open drainage channels in agricultural land drainage systems in 22 countries. Framji et al. (1987) give a similar overview of the design practices for covered drains.

The design of drainage canals is usually determined by the choice of the design discharge, the freeboard that is to be maintained and the necessary assurance that a certain flow velocity is not exceeded. A relatively simple calculation routine, based on stationary equations (Chezy, Manning) leads to the dimensions of the various watercourses in the drainage system. In areas where the land use, soil type, drainage conditions, etc. are very heterogeneous and therefore the rainfall runoff relations can differ considerably, the question arises whether this calculation still gives satisfactory results. Particularly the developments in the extent of the housing and industrial areas and glasshouse areas lead to the introduction of a second calculation step for this kind of drainage area. This second step consists of a check calculation in which the water movement through the drainage system is recalculated according to a routine based on non-steady equations. The application of models describing the non-steady flow in open and closed systems has become much more prevalent because of the greatly increasing use of computers.

The fast working, screening evaluation method, the tool for the designer as introduced in Section 1.3 is basically directed at the optimization of this second calculation step, the cyclic process where the qualities of a variant are evaluated. It is therefore directed at the presentation of the two main aspects: the performance of the system and the economic consequences.

The performance of the system is determined by:

- the hydrologic qualities of the drainage area, i.e. the rainfall runoff relations of the various draining surfaces;
- the hydraulic qualities of the system of watercourses as designed for the underlying variant. The judgement on these qualities should include consideration of the question to which extent the underlying variant meets the design criteria as set a priori by the designer.

The economic evaluation takes into account the investment costs, the maintenance costs, the exploitation costs and, if they occur, the benefits. If benefits are absent the economic evaluation is reduced to the presentation of the net present value of the costs for a given time horizon and rate of discount.

To obtain these evaluation results the EWAS model consists of the following four parts:

- a part describing the non-steady flow in open channels;
- a part with which the drainage discharge caused by rain in the catchment area is determined;
- a part by which the water levels that are realized in a simulation are confronted with the criteria;
- a part to determine the net present value of the costs for all parts of the system as well as for the whole system, or the internal rate of return if benefits are known or can be estimated.

To be more specific about the evaluation procedure a hypothetical example of the results of an evaluation with EWAS is given in Table 4.1. The example applies to a water management system subdivided into 16 branches and 17 nodes. Several criteria are defined in these nodes. In some nodes there is only one set, in other nodes there are two or more sets of criteria. Each set represents the criteria of the future manager for a certain land use and is formulated in two ways: the first is a water level that is not to be exceeded, the second is a water level that is not to be exceeded longer than a given period of time. If a criterion is met, the node scores a plus in the matrix of Table 4.1., otherwise a minus. The significance of stars in a location in the matrix is that for that particular node no criteria concerning that land use have been formulated. The whole matrix of plus and minus reflects to what extent the water management system fulfils the design criteria in terms of water levels.

\ crit. \ ----\ \ node no.\	urban		recreation		nat. res.		agricultural yield [kton]	costs of water management system [mil.Dfl]
	st	nst	st	nst	st	nst		
2	-	+	-	+	+	+	*	0.000
4	-	+	+	+	*	*	*	0.308
6	-	+	+	+	*	*	*	0.000
8	+	+	+	+	*	*	*	0.551
10	+	+	+	+	*	*	*	0.000
12	-	-	*	*	*	*	*	0.000
14	-	+	*	*	*	*	*	0.304
16	-	+	*	*	*	*	*	0.262
18	+	+	*	*	*	*	*	0.201
20	+	+	*	*	*	*	*	0.207
22	+	+	*	*	*	*	*	0.168
24	*	*	*	*	+	+	*	0.434
26	*	*	+	+	+	+	*	0.000
28	*	*	+	+	+	+	*	0.204
30	*	*	*	*	*	*	*	0.192
32	*	*	*	*	*	*	*	0.199
34	*	*	*	*	*	*	*	0.417
total yield [kton]							*	
total costs [mil.Dfl]								3.447
total benefits [mil.Dfl]							*	
Net Present value [mil.Dfl]								3.492
Internal rate of return [%]								*

Table. 4.1. Example of an evaluation with EWAS where the criteria used are related to different types of land use, i.e. urban, recreation and nature reserve areas. The 'st' and 'nst' criteria refer to respectivily the steady and the non steady design criteria specified for that particular land use. The score of a + indicates that the criterium is met, a - indicates the opposite. A * denotes the absence of that type of land use in that particular location. The cost presented in the last column are ment for illustration purposes only.

crit. \ node no.	severe		more severe		severest		costs of water management system [mil.Dfl]
	st	nst	st	nst	st	nst	
2	+	+	-	+	-	-	0.000
4	+	+	-	+	-	-	0.308
6	+	+	+	+	-	+	0.000
8	+	+	+	+	-	+	0.551
10	+	+	+	+	+	+	0.000
12	+	+	-	+	-	-	0.000
14	+	+	-	+	-	+	0.304
16	+	+	+	+	-	+	0.262
18	+	+	+	+	+	+	0.201
20	+	+	+	+	+	+	0.207
22	+	+	+	+	+	+	0.168
24	+	+	+	+	+	+	0.434
26	+	+	+	+	*	*	0.000
28	+	+	+	+	*	*	0.204
30	+	+	+	+	*	*	0.192
32	+	+	+	+	*	*	0.199
34	+	+	+	+	*	*	0.417
total costs [mil.Dfl]							3.447
Net Present value [mil.Dfl]							3.492

Table 4.2. Example of an evaluation with EWAS where the criteria used are related to different return periods of the design conditions. The 'st' and 'nst' criteria refer to respectivily the steady and the non steady design criteria specified for that particular return period. The score of a + indicates that the criterium is met, a - indicates the opposite. A * denotes the absence of that type of land use in that particular location. The cost presented in the last column are ment for illustration purposes only.

The same routine can also be applied in another way. In the absence of well defined criteria for a certain kind of land use there may be some merit in the application of various sets of criteria that are not related to a specific land use as such, but are merely related to different return periods of the design conditions. In Table 4.2 an example is given of such an application. Instead of the land use criteria now the three different sets of severe, more severe and most severe criteria are given. In this case the severest criteria concern the set of criteria with the greatest freeboard which is related to the shortest return period. The set of the severe criteria, with the smallest freeboard, is related to design conditions with the longest return period. It is this last

application of the set of critreria that is used in the case dealt with in Chapter 5.

The above mentioned parts are treated in the following Section 4.2. In each sub section a review is given of present theories and practices, together with the considerations that have led to the choices for the application in the EWAS model.

4.2 Model description

4.2.1 Introduction

The application of models describing non steady flow in open and closed conduits have grown strongly with the growing use of computers. Illustrative in this context is the comparison of the review given by the working group for discharge calculations in 1979 (Werkgroep afvoerberekeningen, 1979) and the review given by SAMWAT based on data of 1988 (Volp and Lambrechts, 1988). Whereas in the first review 10 models for non steady calculations in open conduits are given and 1 model for closed conduits, the second survey reports the existance of 19 models for the description of the water movement in a system of watercourses. Only 4 models are mentioned in both surveys. The growth in development of models is more pronounced if in the second review the selection is broadened to models for the description of transport of solutes, heat and sediment and for the description of the biological processes in conduits. The SAMWAT database then reports the availability of 66 models of which 40 run on personal computers.

The situation with respect to the availability of models in 1983 could very well be described by the situation in 1979. The available models though different in I/O were all tested and validated and gave satisfactory results. The development of EWAS started with a copy of the KNOTA program as developed by Bouman and Schultz (1978). Apart from the theoretical backgrounds and the availability the considerations for choosing KNOTA were rather pragmatic: the program was written in FORTRAN and thus easily extended and/or modified and it was well described.

To perform the specifications as enumerated in Section 1.3 and mentioned in Section 4.1 the following three modules are added to the original flowchart:

- A module for the determination of the drainage discharge;
- A module for testing the water levels that are obtained in a simulation against criteria given by the designer a priori.
- A module for the economic evaluation of the system of watercourses.

In the following sections 4.2.2 to 4.2.5 the backgrounds of KNOTA and of the above mentioned three modules are explained.

4.2.2 Open channel flow

The heart of EWAS is formed by the KNOTA model that calculates the non-steady flow in open channels. The equations of continuity and of flow are solved by an implicit difference scheme. On this subject a vast amount of literature is available. General background information can be found in Stoker (1957), Chow (1959), Mahmood and Yevjevich (1975) and Allersma (1973), while for the computational methods reference is made to Vreugdenhil (1973), Abbott and Ionescu (1967), Chandry and Contractor (1973), Fread (1973), Dronkers (1969). For readers not familiar with this type of model and especially with KNOTA a short description of the basic equations and calculation method is given in this section.

Basic equations

Non-steady flow in open channels can be described by the equations of continuity and motion, the so-called De Saint-Venant equations. By solving this set of equations in combination with initial conditions and boundary conditions, the variation of discharges and water levels in relation to time can be determined for all points of the system. For the numerical solution of the De Saint-Venant equations an implicit difference method has been chosen here. This choice was based on the advantages of the method with respect to the limitations of the time step in connection with the stability of the calculated results. This relates e.g. to those systems of watercourses where the water levels can change rapidly due to rapid response to discharges of rainfall from urban and glasshouse areas. To apply this method, the system of watercourses is divided into branches and nodes. The water levels are determined at the nodes, where the continuity equation must apply (Fig.4.1). For a node this equation can be written as (CHO-TNO, 1973):

$$F \, . \, \frac{h_{t+\Delta t} - h_t}{\Delta t} = \Theta . \, [\, \sum_j Q_j + Q_e \,]_{t+\Delta t} + (1-\Theta) \, . \, [\, \sum_j Q_j + Q_e]_t \qquad (4.1)$$

where: F = open water area of the node $[m^2]$

h = water level $[m]$

t = time parameter $[s]$

Δt = time step $[s]$

Θ = time centre coefficient $[-]$

Q_j = inflow from the connected branch j $[m^3.s^{-1}]$

Q_e = lateral inflow being the sum of rain and runoff and drain discharge $[m^3.s^{-1}]$

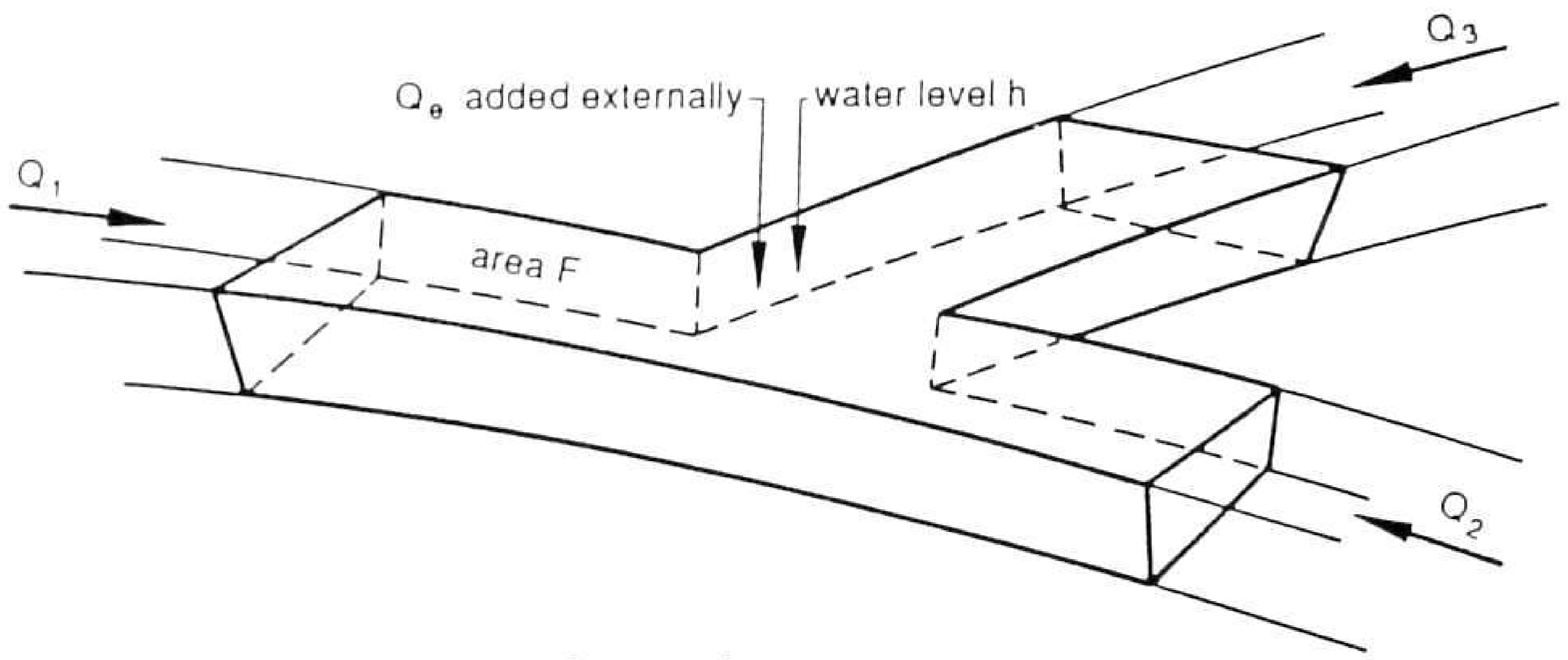


Fig. 4.1. Schematization of a node.

The discharges are determined using the equation of motion for the branches (Fig. 4.2.). This equation for a branch is:

$$\frac{L}{g.A} \, . \, \frac{Q_{t+\Delta t} - Q_t}{\Delta t} - \frac{Q.B.L.}{g.A^2} \, . \, \frac{h_{1,t+\Delta t} + h_{2,t+\Delta t} - h_{1,t} - h_{2,t}}{\Delta t} +$$

$$\Theta \, . \, [(1-Fr^2).(h_2-h_1) + \xi.Q. \, Q^{\cdot}]_{t+\Delta t} +$$

$$(1-\Theta) \, . \, [(1-Fr^2).(h_2- h_1) + \xi.Q \, Q \,]_t - I.L. = 0 \qquad (4.2)$$

where: L = length of the branch $[m]$

g = acceleration due to gravity $[m.s^{-2}]$

A = flow area $[m^2]$

Q = discharge $[m^3.s^{-1}]$

t = time parameter $[s]$

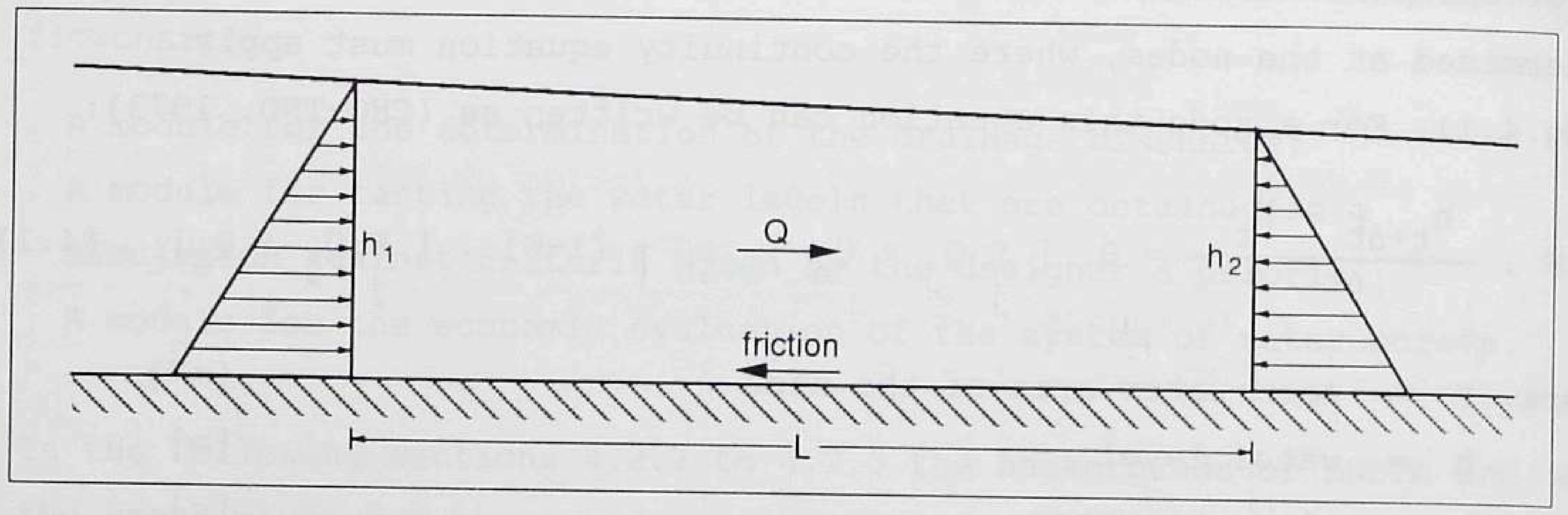


Fig. 4.2. Schematization of a branch.

Δt = time step [s]

B = width at the waterline [m]

$h_{1,2}$ = water depth at the upstream (1) or downstream (2) end of the branch [m]

Θ = time centre coefficient [-]

Fr = Froude number [-]

$$Fr^2 = \frac{Q^2.B}{g.A^3} \qquad (4.3)$$

ξ = friction loss coefficient $[m^{-5}.s^2]$

$$\xi = \frac{L}{Ch^2.A^2.R} \qquad (4.4)$$

where: Ch = Chezy friction coefficient

$$Ch = 1.5.R^{1/6} . {}_{m} \qquad (4.5)$$

where: k_m = Manning friction coefficient $[m^{1/3}.s^{-1}]$

R = hydraulic radius [m]

I = bottom slope [-]

The time centre coefficient determines the resemblance of the calculated values to the real (measured) values and has an important influence on the stability of the calculation process. The best resemblance is obtained for the value Θ = 0,5; for this value, however, the chance of instability is greatest. The value Θ = 1.0 causes the calculated values to be attenuated compared to the measured values. The calculation process with this value is very stable. This equation of motion can be extended by including the effects of wind forces and/or modified if the branch contains structures such as weirs, culverts or pumps. For these equations the given references may be turned to. The final shape of those equations is basically of the same structure as the equation of a branch, or can be so written. The above equations 4.1 and 4.2 are transformed, rewritten and linearized by neglecting where necessary the terms with a power over one. Rewriting the equation of continuity so that the water level changes are expressed in terms of changes in the inflowing discharges and using this equation in the equation of motion, an equation is found in which the only unknowns are the changes in the discharges of the connected branches. For a system with n branches a set of n linear equations with n unknown discharge changes is formed which can be solved for every time step. Using the results in combination with the boundary conditions in the equation of continuity yields the new water levels.

4.2.3 Discharge of rainfall

Introduction

In the determination of the dimensions of the watercourses of a drainage system the magnitude of the discharge of the rainfall on that area plays an important role. There are two main different approaches. For the first step of the dimensioning process, as mentioned in Section 4.1, which is a stationary consideration, the drainage coefficient is generally applied. The value of this coefficient may vary from 0.3 to 7.0 $l.s^{-1}.ha^{-1}$ depending on meteorological conditions, average slope of the land, drainage density, soil type, crop type and return period (Smedema and Rycroft, 1983; Segeren and Luijendijk, 1982). For application in EWAS, which is especially directed at the second design step where the considerations mainly concern the non-steady effects, the drainage coefficient will not be satisfactory. The runoff process must therefore be described in greater detail. The schematization in nodes and branches, necessary for the determination of the flow through the watercourses, offers the opportunity to concentrate on the determination of the discharge of rain fallen on the sub-catchment area around a node. In describing the runoff process the reasoning can then be restricted to the process from interception to discharge into the watercourses which are part of the schematization. For application in the EWAS model the formulation of the runoff process should offer the opportunity to define different transformations of the rainfall into runoff for each node, in order to gain maximum flexibility without too much effort.

To arrive at design discharges from rainfall, decisions have to be made with respect to: the gross rainfall, the rainfall losses and the transformation of the rainfall hydrograph into the runoff hydrograph. In the following sections the above subjects will be discussed in more detail.

The gross rainfall

There are basically two ways to determine the design conditions originating from the rainfall. The first and most recommended way, but which is also the most elaborate and most expensive one, is to execute all calculations for a long historical series of meteorological data. After all calculations have been carried out, the design conditions can be determined from a statistical analysis of the results. The meteorological data must be available for a sufficiently long period of time and contain enough details with respect to the variation in time of the rainfall intensity. The main advantage of this method is that information is obtained about the results for events more severe than the design conditions.

Although it is generally acknowledged that this procedure yields the best results with respect to design conditions and return periods, it is seldom applied, for two reasons. First, the meteorological data covering the requirements both of time length and of detail are seldom available. Secondly, if the data are available, the costs related to the execution of the calculations, especially for the larger systems, will be a major obstacle. A selection of the recorded data with respect to certain characteristics can be a partial solution to the latter problem. Selection criteria depend on the characteristics of the runoff surfaces.

In cases where it is impossible or too expensive to execute all calculations for reasons outlined above, the most commonly applied solution is the definition of a design rainfall, which is rainfall with a certain volume over a given period of time with a return period of T years. The resulting design discharges in this reasoning are assumed to have the same return period. Much research work has been done into the development of design rainfalls for the design of storm sewer systems, obviously because of the very great investments involved. The design rainfall is usually derived from depth duration frequency relationships. The depth duration frequency curves (DDF-curves) are the result of a statistical analysis of single independent rainfalls. For each rainfall event the maximum rain volumes for n different durations are evaluated and the frequency distribution is determined for each of the n durations. Combining the results for specified frequencies in one graph results in the DDF-curve for a given return period.

Choosing a design storm from this DDF-curve is most commonly done by exploring the whole curve in search of the one event that results in the highest discharges or the highest water levels. This search is carried out for more than one variation (chosen a priori) of the rainfall intensity during the event, because this variation may have a considerable effect on the outcomes especially if applied to fast reacting surfaces like glasshouses and parking lots. Two considerations apply with respect to the use of design storms derived from DDF-curves concerning respectively the variation of the intensity during the event, and the rainfall volume of the total rainfall event.

As to the variation of the rainfall intensity during the rainfall event it should be realized that the DDF-curve contains information about the total volume of rain fallen within a certain time interval and about the maximum intensity only. It does not give any information about the variation of the rainfall intensity during that time interval. Various researchers have elaborated on this subject. Here van de Ven (1983,a,b) and Pfeiff (1971) should be mentioned. Ven demonstrated the consequences of a choice of the storm shape with respect to the return period. The location of the peak period within the storm period can vary, and its 'most critical' location depends on the system under consideration. The choice of this location is, however, not completely arbitrary as Pfeiff has shown. Pfeiff (1971) analysed rainfall data for Ludwigshaven and found that the maximum of rainfall intensity mostly occurs within the first third part of the total rainfall time period or, to a lesser degree, in the last third part of the rainfall time period. In addition he found this to be a function of total rainfall volume. The larger this volume, the less definite the location of this peak intensity is fixed, but there is a preference for the first two third parts of total rainfall time period.

The second aspect concerns the rain volume of a storm derived from a DDF-curve. The rain volume of such a storm represents only a part of the total volume of the real rainfall event. The 'volume' fallen prior to and directly after the considered duration is not included in the analysis. Especially the volume that has fallen prior to the storm is of great importance because of its influence on the rainfall losses that have to be taken into account to arrive at the net rainfall. Whether the runoff surfaces are already wetted, the depressions (partially) filled and the infiltration started or not, these circumstances are clearly of great importance for the determination of the losses on the rain volume of the storm under consideration. In order to take this problem into account, Sifalda (1973), suggested adding rainfall prior to and directly after the design rainfall derived from a DDF-curve. Based on an analysis on rainfall data of some places in Czechoslovakia he gives rules for the derivation of these additional rainfalls.

The above treatise on gross rainfall is given because in the case described in Chapter 5 design storms are chosen from a DDF-curve as outlined at the beginning of the section. That case serves to demonstrate that the choice of the design storm with respect to rainfall volume and variation of the rainfall intensity is strongly related to the kind of land surface of the catchment area.

The rainfall losses

The concept of the rainfall losses within the framework of rainfall runoff relations can be defined as the difference, in volume and in relation to time, between the gross rainfall, as recorded with rain gauges, and the corresponding discharge, thus reducing the gross to the net rainfall. The difference occurs because a part of the rainfall is either intercepted and evaporated as for instance in depressions on the soil surface, or it replenishes the various storages as for instance in the unsaturated and/or saturated soil. The volume of the differences depends on the state of the area under study. This state is determined by the various physical qualities such as the overgrowth, soil, soil moisture content, infiltration capacity, drainage conditions and the depth of the groundwater table, and is to a great extent determined by the weather conditions in the preceding period. It should be noted here that the method applied to determine the rainfall losses strongly depends on the case at hand, the availability of data and the choice made as to whether a succession of historical storms is considered or the idea of a design rainfall is adopted. For reasons of flexibility, for EWAS the choice was made to work with the net rainfall data.

The transformation of the rainfall hydrograph into the runoff hydrograph

Following the course of the water particles the subject that remains is the description of the transformation from net rainfall hydrograph to the resulting discharge hydrograph. This transformation process has been described by many authors. Reviews of theories and practices are given by Linsley et al. (1958), Chow (1964), Soil Conservation Service (1964) and Bureau of Reclamation (1974). Developments in the Netherlands were especially directed at the description of the relatively slow reacting groundwater reservoir (Kraijenhoff,1958; Hellinga and Zeeuw, 1958; Jager, 1965; Zeeuw, 1966; Ven, 1979). General applicability of the Kraijenhoff model has been reported by Zeilmaker (1973). Nes (1973) presented a linear-distributed model of surface runoff based on the simplified and linearised one-dimensional equation for unsteady flow in a channel. Zondervan (1978) presented a quasi - linear approach, Ven et al. (1981) evaluated the performance of various types of models with data collected in Lelystad (the Netherlands).

Depending on the case under study, the transformation model must account to a greater or lesser extent for the various storage possibilities that the water encounters on its way to the drainage system. In dealing with the selection problem of finding an appropriate model to use in EWAS there are five criteria to consider:

- simplicity of the model;
- flexibility of the model for various kinds of watersheds;
- consistency of parameter estimates;
- sensitivity of the results to changes in parameter values;
- accuracy of the description of the process.

Simplicity refers to the number of parameters that must be estimated and the ease with which the model can be explained to clients or public bodies. All other factors being equal, the model with the minimum number of parameters should be chosen. Flexibility refers to the possibility of using the same model with different parameter values for different kinds of watersheds, urban and agricultural, for instance. Consistency of parameter estimates refers to the sensitivity of the parameter values to different periods in the same watersheds or to the question whether the parameter values vary widely between similar watersheds. Sensitivity refers to the sensitivity of the results to those input variables that are difficult to measure. Finally, models should give a reasonably accurate prediction of the system outputs. Again, all other factors being equal, the model with the minimum of bias and error variance is superior. As Haan et al. (1982) state, the above criteria are related, but have to be considered when a model is being chosen. The final choice of the 'best' model will depend on the problem at hand, the resources available to the analyst, the time frame available and on a number of other implicit criteria such as experience, feeling and intuition.

The most important criteria in selecting a transformation model for EWAS, are the model simplicity and flexibility because they concern the general applicability of a model. The other criteria, consistency, sensitivity and accuracy relate to the parameter values determined in other research projects. For EWAS it is only of interest that parameter values exist for different kinds of watersheds. Another important factor is that in the application in EWAS the total surface area under consideration, the 'watershed' around a node, is relatively small. Furthermore the configuration of branches and nodes should preferably be chosen in such a way that the 'watersheds' are homogeneous with respect to the runoff characteristics.

Considering the above, the rainfall runoff model to be implemented in EWAS should be able to describe the response of such a small catchment area possibly with a slow and a quick reacting component in combination with a constant base flow. The model should furthermore be formulated in general terms for general application. This is realized in EWAS by defining the runoff of the sub-catchment area of a node by the sum of the discharges of two cascades of linear reservoirs, a base flow and the rainfall directly into the open water surface:

$$Q_{e,i,t} = q_{p,i,t} \cdot B_i \cdot p_{p,i} + q_{u,i,t} \cdot B_i \cdot p_{u,i} + d_t \cdot (B_i + F_{i,t}) + P_t \cdot F_{i,t} \qquad (4.6)$$

where: $Q_{e,i,t}$ = total lateral inflow towards node i at time t [$m^3.s^{-1}$]

$q_{p,i,t}$ = specific discharge resulting from rainfall on paved area in the catchment area of node i at time t [$m.s^{-1}$]

$q_{u,i,t}$ = specific discharge resulting from rainfall on unpaved area in the catchment area of node i at time t [$m.s^{-1}$]

B_i = total surface area of the catchment area of node i, open water area excluded [m^2]

$p_{p,i}$ = percentage paved area of F_i [-]

$p_{u,i}$ = percentage unpaved area of F_i [-]

P_t = rainfall at time t [m]

$F_{i,t}$ = total open water area of node i at time t [m^2]

d_t = base flow at time t [$m.s^{-1}$]

The specific discharges $q_{p,i,t}$ and $q_{u,i,t}$ are determined in EWAS as a resulting discharge from a Nash cascade where rain falls in the first reservoir only. The number of reservoirs and the reservoir coefficients may vary for every node. It is clear that other rainfall runoff models can be implemented easily if desired or in cases where calibrated runoff models are available. In simulations carried out with EWAS so far, only the above routine was used and it proved to be simple, flexible and easy to work with. The routine, with well chosen parameters, gave adequate results while simulating runoff that could be verified against measured data.

4.2.4 Design criteria

In the light of the discussion of Section 1.2 the concept 'design criteria' comprises the hydraulic quality criteria as well as the set of criteria provided by representatives of a certain land use, in absence of the explicit relationship of water management and benefits.

Assuming the economic value to be the same, the freeboard to be maintained, in case of a design discharge with a recurrence period of 5 years will be larger than the freeboard in case of a design discharge with a recurrence period of 25 years. A study of the reports for various countries reveals according to Framji et al. (1984), that the minimum freeboard is taken as zero, if mentioned at all. In most reports the freeboard is not mentioned but implicitly taken as zero by wording like: "flooding is to be avoided as much as possible". The recurrence period varies between 5 and 15 years, depending on the economic value of the area. In a number of cases a period of inundation is accepted. The depth and the period of time of inundation is related to the economic consequences of the flooding. This is illustrated by the requirements as reported for Australia, and is valid for irrigated areas. The Australian requirements for the period of inundation depend on farm sizes, accounting for the scale effects of a given loss in benefits, on the kind of crops, accounting for the tolerance of the crop with regard to inundation, and on the economic value. Besides, there is a limit to the maximum area which may be inundated: 10 and 30 percent of gross farm area for small and large farms respectively, with design frequencies of 1 in 10 years. Smedema and Rycroft (1983) give guidelines for the freeboard depending on the size of the canals: a freeboard of 30 cm for the small canals ($Q < 1\text{-}2\ m^3.s^{-1}$) and a freeboard of 50 cm. for the large canals ($Q > 5\ m^3.s^{-1}$).

In the design of the drainage system in agriculture areas in the Netherlands three different levels are defined (Werkgroep Afvoerberekeningen,1979):

. Normal Water level (N.W. level). This level is reached or exceeded on 10 to 20 days a year;
. High Water level (H.W. level). This level is reached or exceeded on 1 or 2 days a year;
. Maximum Water level (M.W. level). This level is reached or exceeded once or twice in 100 years.

The relation between these three levels and the corresponding discharges and water depths is given in Table 4.3. for a free discharging channel system with bank slopes of 1:1½. The above three levels were defined for arable or grass land areas for different soil types with respect to the drainage base. The H.W. levels are 20-30 and 30-40 cm above the drainage base for grassland and arable land respectively. For the N.W. levels these figures are 5-20 and 15-20 cm. The freeboard in the case of the H.W. levels is 50-60 cm for both the arable and the grassland for all soil types. The freeboard in the case of the N.W. levels varies from 70 to 130 cm with land use and soil type. The basis of these criteria is the expected loss of yield due to too wet conditions in the upper layers of the profile, these conditions being associated with poor workability and poor aeration. The relation between the degree to which the above levels are exceeded and the real loss of yield is discussed in Section 3.3.

Table 4.3. The relation between various design levels, discharges and water depths (Werkgroep Afvoerberekeningen, 1979).

level	discharge	water depth	frequency of occurrence
N.W.	Q_n	h_n	10 to 20 days per year
H.W.	$Q_h - Q_n * 2$	$1.4 * h_n$	1 to 2 days per year
M.W.	$Q_m - Q_n * 4$	$2 * h_n$	1 to 2 times per 100 years

The design criteria in terms of water levels of watercourses in other than agricultural areas are not as clearly described as the above. This is due to lack of knowledge about the relation between water level and specific land use. This will be clarified in the following for urban areas and nature reserve areas.

A commonly applied design rule for the drainage system in urban areas is that the reference level may be exceeded by about 0.20 to 0.50 m with a frequency of occurrence of once per 10 years. The final design is checked for design conditions with a frequency of occurrence of once per 2 years and once per 100 years. The damage that results from these events depends on the topography: the low-lying areas will be inundated first and most severely; and the land use: the damage expressed per unit area, per unit time and per unit depth that will occur, depends on the value of the investments made for the development of the area.

To evaluate the seriousness of the damage, three classes of damage can be distinguished: the direct, the indirect and the secondary damage. The direct damage is the one originating from the contact of the water with the goods that can be affected. The quantification of the damage is usually based on the costs for repair or replacement. In urban areas, gardens and parks will suffer damage, primarily due to higher groundwater levels, but also due to inundations. Parking lots, roads, bridges and tunnels may be damaged by inundations. But such damage will be slight compared to the damage that will occur in industrial and housing areas. In industrial areas it can vary widely, depending on the kind of industry, the sensitivity to flooding of stocks and the value of the product. In areas with houses, the type and age of the house and its furnishings will determine the damage. Griggs and Helweg (1975) give relations for the depth of inundation and the resulting damage, as a percentage of the total value of the house and its contents. Other more detailed relations are given by the working party on hydraulic design of storm sewers (1981) for various types of houses, offices, shops and industries.

The indirect damage consists of the value of: lost services, working hours, industrial production; the costs of neutralizing inconveniences; the consequences of obstruction of traffic facilities. This damage is very hard to quantify. Literature data provide estimates related to the direct damage, differentiated for various land uses (Grigg and Helweg, 1975).

To quantify the secondary damage is just as difficult as it is to quantify the indirect damage. The secondary damage consists of the damage suffered by others than those directly involved and the relatively intangible damage such as that affecting the quality of the environment. Disadvantages of using these figures are due to the fact that they are only indirectly a function of the depth and the duration of the inundation.

The above outline shows that much of the damage that will occur in urban areas is very hard to quantify. Hence the criteria for the permissible water levels are based on other grounds, such as the assumption that all roads and paths and outlets of subsurface drainage should remain dry. In this approach it is the public works department which sets the criteria, based on the lowest points of roads, outlets and the like in the affected area around each node in the schematisation. These criteria may then be subject to discussion during the design process.

A similar approach appears to be the most appropriate for other kinds of land uses such as nature reserve areas. Due to the fact that the many relationships between fauna and flora on the one hand, and surface water and groundwater, including water quality on the other hand, are still subject to research, the approach described above is in fact the only approach. In this case too the initial criteria may be evaluated during the design process, taking into account the results of the simulations.

4.2.5 Economic evaluation

As stated in Section 1.2 the function of the economic evaluation in the

EWAS programme is only to supply a criterion for the comparison of the various variants. For that purpose there is no need for an extensive project evaluation as used for example in the HELP evaluation method (Werkgroep Herziening Evaluatie Landinrichtingsplannen, 1982). Bosma (1986) describes the HELP project evaluation method in his dissertation, discussing all elements included. HELP was designed as an evaluation tool for land development projects from a national viewpoint. The evaluation is not restricted to economic factors only. In addition to social aspects the effects to nature, landscape and the environment are also included. Because of the restricted function of the evaluation in the EWAS application programme it can be limited to an economic evaluation. The EWAS programme may serve within the designing process of the system of watercourses of a particularly land development project to develop variants that will be taken into consideration in the much wider scope of the HELP evaluation method. Bosma states that after some years of experience with the HELP method, even this detailed evaluation method is used for carrying out the process of 'evaluating during designing'.

In the case at hand the economic evaluation must clarify the financial consequences with respect to the water management and the system of watercourses. In the beginning of this research project the thought prevailed that the economic evaluation could involve the costs as well as the benefits. This would have resulted in the ability to determine the internal rate of return on condition that agricultural benefits would be the only benefits to consider. The PRODU model was developed for that purpose, i.e. to be able to establish the relation of agricultural benefits and water management. The concept 'water management' was considered to include the total number of measures with respect to the management of water levels: the choice of the (summer and winter) levels, the switch dates from winter to summer level or vice versa, the storage capacity, the transport capacity and the control mechanism. An important result of the experiences with the PRODU model is that for given drain depths and drainage intensities the choice of the levels does not influence the potato yield very much if extreme situations are not considered.

Because the relation described above could not be established, the economic evaluation can be restricted to the determination of the net present value of the costs for construction, maintenance and exploitation of the system of watercourses. This value can be determined from:

$$C_s = C_{s,g} + \sum_{i=1}^{n} C_{b,i} \qquad (4.7)$$

where: C_s = net present value of the costs of the whole system [Dfl]

$C_{s,g}$ = net present value of the general costs for the whole system [Dfl]

$C_{b,i}$ = net present value of the cost for each branch i [Dfl]

n = total number of branches [-]

The concept of the general costs comprises the costs incurred by the Waterboard which can be ascribed to that particular area (obviously, only the water management costs are considered). Examples of this kind of costs are administrative expenses, salary costs and housing costs. This item is included because part of the comparison may be that another type of water management generates smaller general costs. The question arises whether it will be possible, in the Waterboard practice, to estimate the rise or reduction of these costs in relation to the chosen type of water management. Because these costs are incurred every year they can be determined by:

$$C_{s,g} = \sum_{t=1}^{N} \frac{c_{s,g}}{(1+r)^t} \qquad (4.8)$$

where: $C_{s,g}$ = net present value of the general costs for the whole system [Dfl]

$c_{s,g}$ = yearly general costs for whole system for the Waterboard [Dfl]

r = rate of discount [-]

N = Life time project [years]

The costs per branch can be determined by:

$$C_{b,i} = c_{I,i} + \sum_{t=1}^{N} \frac{(c_{E,i} + c_{M,i})}{(1+r)^t} \qquad (4.9)$$

where: $C_{b,i}$ = net present value of the cost for each branch i [Dfl]

$c_{I,i}$ = investment cost for branch i [Dfl]

$c_{E,i}$ = yearly energy costs for branch i [Dfl]

$c_{M,i}$ = yearly maintenance costs for branch i [Dfl]

r = rate of discount [-]

N = life time of project [years]

The expressions of equations 4.8 and 4.9 can very well be extended by also considering items like (re-)construction period, yearly rise in construction costs, inflation and yearly rise in energy and maintenance costs. In view of the purpose of this routine the latter items are not taken into consideration in favor of the more simple approach. To be able to determine the C_s if it concerns the rehabilitation or the reconstruction of an existing system of watercourses, the expression 4.9 is extended by including the value of the existing system:

$$C_{b,i} = c_{I,n,i} - c_{I,o,i} + \sum_{t=1}^{N} \frac{(c_{E,i} + c_{M,i})}{(1+r)^t} \qquad (4.10)$$

where: $C_{b,i}$ = net present value of the cost for each branch i [Dfl]

$c_{I,n,i}$ = investment cost for branch i [Dfl]

$c_{I,o,i}$ = investment cost for branch i with the actual dimensions [Dfl]

$c_{E,i}$ = yearly energy costs for branch i [Dfl]

$c_{M,i}$ = yearly maintenance costs for branch i [Dfl]

r = rate of discount [-]

N = time horizon [years]

In the following the investment costs as well as the costs for energy and maintenance per branch are discussed in more detail.

The investment costs

The investment costs per branch are determined by using prices per unit for the various kinds of investment cost that can be distinguished:

. land purchase	$[\mathrm{Dfl.m^{-2}}]$
. surveying costs	$[\mathrm{Dfl.m^{-1}}]$
. excavation costs	$[\mathrm{Dfl.m^{-3}}]$
. costs for slope protection	$[\mathrm{Dfl.m^{-1}}]$
. costs for finishing the maintenance path	$[\mathrm{Dfl.m^{-2}}]$

If structures are to be built in the branch under consideration the investment costs of these structures are added to the investment costs of the branch. It turned out to be difficult to work with price per unit for these structures because the foundation costs appeared to be an important factor depending on the situation on the spot. The approach of working with separated items for foundation costs and other construction costs is adopted here.

The value that must be used for these items will depend on the local prices in a particular case. In the Netherlands a workable first approach is to use the prices that are published yearly in a standard price list by the Land Development and Consolidation Department of the Ministry of Agriculture (LD, 1987). For the investment costs of pumps real estimated cost can be used. For this purpose the programme offers the possibility to use Costs Estimation Relationships (CER) developed in the framework of the PAWN studies (RAND Corporation, 1981-1983) to determine the investment costs as well as the yearly energy and maintenance costs.

The energy costs

The energy costs of pump and for manipulation weirs must be estimated or can be determined for the pump using the CER's mentioned above.

The maintenance costs

The maintenance costs are an important item in the economic evaluation. The importance is clearly shown by realizing that according to the annual reports of the Dutch Waterboards the ratio of costs for depreciation and interest, maintenance and administration and management are 30, 55 and 15 % respectively, with a total budget of about Dfl. 400 million. The Waterboards in the Netherlands are responsible for the maintenance of about 53.000 km. of watercourses of different sizes (Loorij, 1979). In this figure is not included the length of channels which the landholders in the polders have to maintain. The best estimate of the total length of channels which have to be maintained varies in lengths of 70.000 to 100.000 km. (Loorij, 1979) up to 150.000 km. (Jager,1983). The total yearly maintenance costs depend on the method of maintenance that is applied and the frequency of maintenance.

The total maintenance costs comprise costs for the removal of the overgrowth on the side slopes as well as on the bottom, the so called small maintenance, and the costs for reshaping the channel, extensive maintenance, where the sediments are removed and the side slopes are reshaped.

About the method and the costs for extensive maintenance not very much can be said. It strongly depends on the case at hand. Important aspects are the quantity of soil to be removed, the method applied for removal, whether the soil can be spread on the adjacent plots or that it must be transported to be used elsewhere or to deposits, the occurrence of polluted components in the outcoming mud etc. The uncertainty and the irregularity in place as well as in time and thus the impossibility to make a prediction of this kind of maintenance cost have been the reason to renounce the including of this kind of cost in EWAS, notwithstanding the fact that the cost will not be insignificant as such.

Methods applied for small maintenance

The following methods can be distinguished:

. physical weed control: weed burning;
. mechanical weed control: cutting and removing of the weeds and clearing the channels of sediments;
. chemical weed control: killing the weeds by applying chemicals;
. biological weed control: shading the channels with trees; application of floating leaves of certain species to prevent or to restrict weed development; and introduction of weed eating fish.

Physical weed control, mainly applied in the Netherlands for the smaller drainage canals, has several disadvantages. The most important ones are the increased risk of erosion damage of the side slopes because the stabilizing effect of roots partly disappears, and the ecological effects caused by killing of the flora on the slopes and a possible decreased oxygen demand in the water as a result of disintegration of the dead organic material. Mechanical weed control ensures the best flow conditions and is the most favourable method from an ecological point of view if applied with care. Disadvantages with respect to costs are decreasing with the development of new machinery and the adjustment of the shape of the watercourses to this new machinery. In 1976 chemical weed control was applied for aproximately 14.000 km of channels for various parts of the slopes (CBS, 1976). Although the results obtained with this method were satisfactory as far as the flow conditions were concerned, most Waterboards have dropped this method because of its ecological effects. Apart from this aspect the method may entail the same disadvantages as does the physical weed control method. Biological weed control turns out to be insufficient to maintain the channel system if applied exclusively. If applied in combination with mechanical weed control methods it can yield satisfactory results. Experiments with the application of water plants to diminish weed growth by means of light reduction indicate that this method, even if applied in combination with weed eating fish, results in rather high roughness values (Pitlo, 1979). Prevention of weed growth by shading of the channels using trees and shrubs on the banks is not economically feasible, because of the larger area necessary. If applied to smaller, less important channels the method is worth considering, especially because of its positive effects from an ecological point of view (Geraats, 1982). The introduction of weed eating fish can yield satisfactory results if conducted with care and if the results are not expected within the first (two) years (Zweerde, 1983). Which method will be applied depends stronly on the weed varieties, the climate, intensity of land use, the level of prosperity and the importance of the channel under consideration. About the costs of the various methods of maintenance very little is known. From the various investigations it is known that the maintenance costs depend on the soil type, the bank slope, the weed varieties, the existence of a maintenance path etc. and varies strongly even within a small country like the Netherlands. As an approach in this problem the same standard price list as mentioned before can be used as a first estimate.

Frequency of maintenance

The costs of maintenance being known per unit length, the necessary frequency is an important factor in obtaining figures for the yearly costs for maintenance. This frequency depends on the natural variation of the roughness coefficient during the year and on the roughness coefficient minimally required during certain periods of the year. The variation, caused by the development of the weeds on the slopes and bottom of the channel is related to the water depth, time of the year and the species of the weeds. Pieters and Flach (1966) and Flach (1967) give graphs demonstrating the variation of the roughness coefficient during the growing season

The yearly maintenance costs for a branch are then determined by the costs per unit length and the frequency of maintenance. As to the maintenance costs for the structures, cost estimates can be derived from the standard list of prices and the CER for pumps.

CHAPTER 5 THE ERICA GLASSHOUSE AREA

5.1 Introduction

To illustrate the application possibilities of the EWAS model, the case of the Erica glassshouse area will be treated in this chapter. The model EWAS should preferably be used in those cases where the non-steady flow and the economical consequences of the (re-)design are important. Both aspects are important in the Erica glasshouse area. The problem of the expanding area of glasshouses in what originally are arable and grassland areas, is of growing importance in many places in the Netherlands. The 'Westland', a polder area in the western part of the Netherlands is famous for this. The need for intensification in agriculture results in an increasing conversion to glasshouse horticulture by farmers in other regions as well. Apart from the increasing area of glasshouses, a second tendency results in a concentration of the glasshouse areas in large and unbroken complexes for reasons of economy and areal planning. It is the changing rainfall-runoff characteristic of the area which causes problems *if the water management system is not adjusted appropriately.*

In Section 5.2. the area and its historical development are described as well as the bottlenecks for the drainage process. In Section 5.3. the modelling of the area with the schematization of the watercourses, the calibration and the validation of the model is treated. In Section 5.4. the choice for the design rainfall and the design criteria is considered, the variants are introduced and evaluated. In the last section of this chapter the EWAS model is discussed against the backgrounds of the experiences described.

5.2 Description of the case

Description of the area

The Erica glasshouse area is located in the north-eastern part of the Netherlands in the region of Emmen in the peat district of south-east

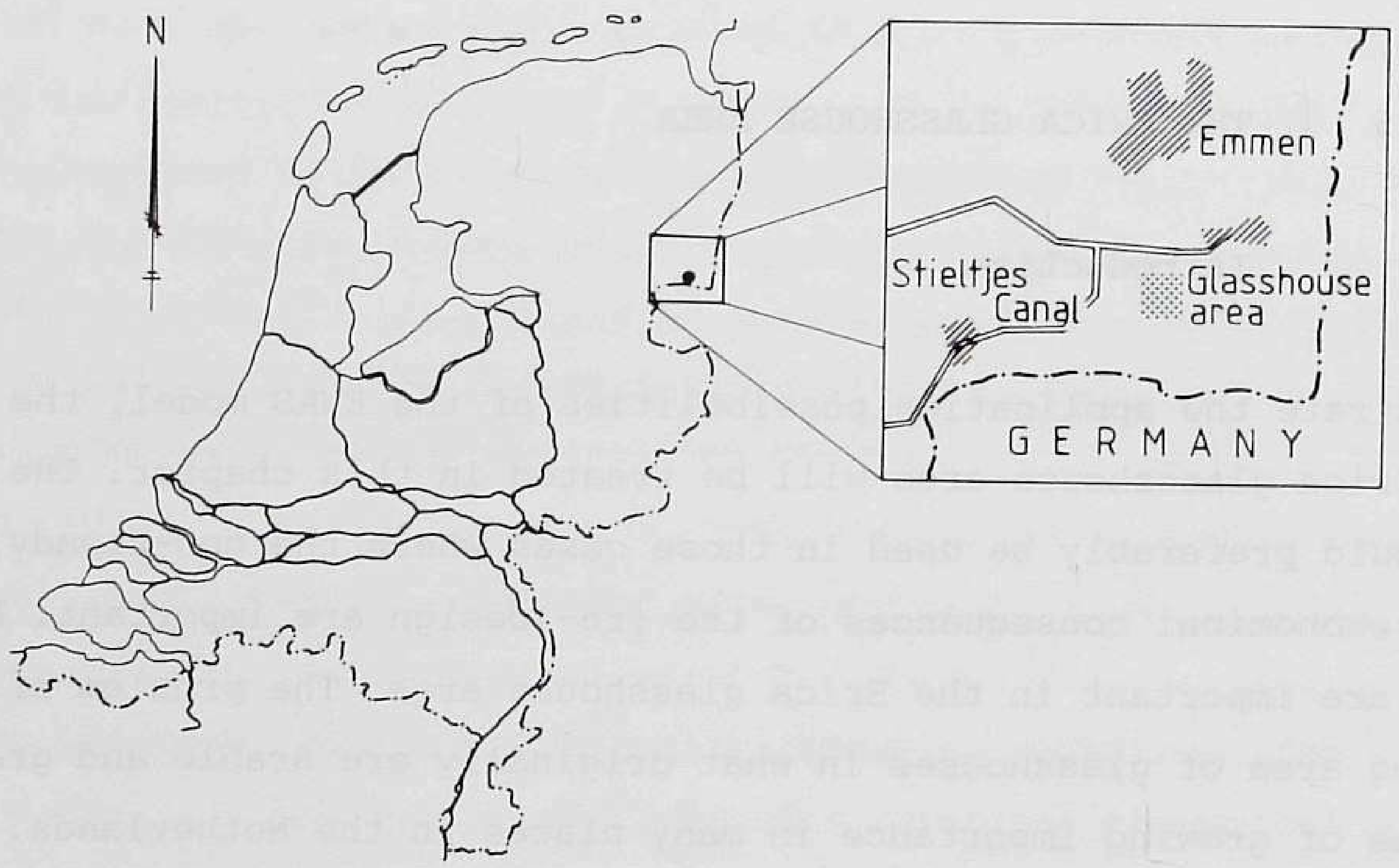


Fig.5.1. The location of the Erica glasshouse area in the north-east of the Netherlands.

Drenthe (Fig.5.1). During the years before the second world war, peat was cut here on a large scale. The remaining soil profile is characterized by a thin layer of sandy peat on sand. After soil improving measures and improved drainage, it is well-suited for agriculture and horticulture.

In Figure 5.2. the drainage situation of the area is given in more detail. The glasshouse area is located in the middle of the figure, south-west of the village of Erica; it consists of five sections: Erica Ik (small block), Erica Ig (large block), Erica II, Erica III and Erica IV. These sections drain into the Central Drainage Canal and differ from each other in, amongst others, land use, drainage conditions and target level of surface water. Erica Ik is an old and compact glasshouse area with minimal freeboard and rather small drainage channels. The drainage of the rain water into the Central Drainage Canal takes place via a culvert upstream from weir S73. Erica Ig is a larger and newer glasshouse area. Here too, there is minimal freeboard and the drainage channels are narrow. As with Erica Ik, Erica Ig drains into the Central Drainage Canal via a culvert upstream from S73. Erica II is a glasshouse area dating from the seventies with a much larger freeboard. The area drains into the Central Drainage Canal via weir S72. Erica III situated in the middle of Figure 5.2., is an area intended for future glasshouse construction.

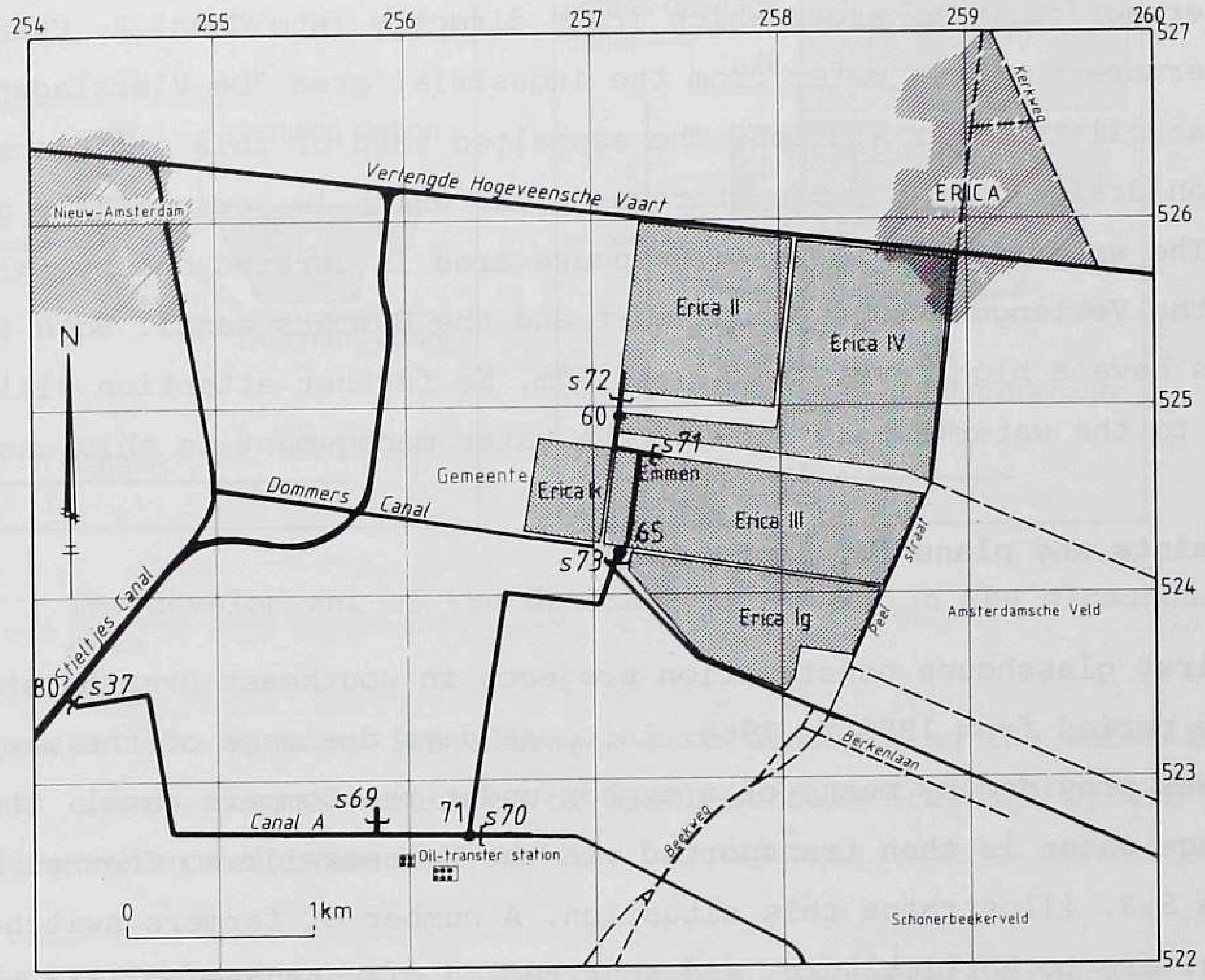


Fig.5.2. The Erica glasshouse area with the structure of the main drainage channels.

Several glasshouses have already been completed. Excess water is discharged into the Central Drainage Canal. To avoid water management problems in future, with respect to water quantity, the construction of the glasshouses is bound by the Waterboard, to the condition that a storage pond, measuring 7.5% of the glasshouse area, will be made in conjunction with the glasshouse. The fifth section of the area, Erica IV, consists of agricultural land. This land is situated to the east of Erica II and III and drains into the Central Drainage Canal via weir S71.

Weir S73 divides the main drainage channel into two parts. The upstream part, the Central Drainage Canal, carries the surplus water from the areas described above and has a target level of NAP +11.20 m. The downstream part, the External Drainage Canal transports the water to Canal A. Immediatly downstream from S73 there is an inverted syphon under the Dommers canal. The water flows via the External Drainage Canal and Canal A to weir S37, which forms the link with the Stieltjes Canal.

Apart from the water from the glasshouse area, Canal A also receives water from two large agricultural areas via weirs S70 and S69 and some smaller agricultural areas which drain directly into Canal A. Channel A furthermore receives water from the industrial area "De Vierslagen" and from an oil-transfer station. The asphalted yard of this oil-transfer station drains into a large storage cellar which is periodically pumped dry. The water supply to the glasshouse area is carried out by inlets from the Verlengde Hoogeveense Vaart and the Dommers canal. Both these canals have a high level of NAP +12,95m. No further attention will be given to the water supply side of the water management in this case.

Complaints and plans for improvement

The first glasshouse construction projects in southeast Drenthe started in the period from 1950 to 1960. In those days drainage of the whole area was provided by means of a syphon under the Dommers canal. The drainage water is then transported via the Meineszwijk to Channel A. Figure 5.3. illustrates this situation. A number of farmers switched from agriculture to horticulture, and constructed glasshouses on land allocated to this purpose by the Council of Emmen, near Erica. To provide a well drained area, the sub-surface drainage system was reconstructed. In addition, the drainage of the area under consideration (Erica Ik and Ig)

Canal A near the National Peat Museum

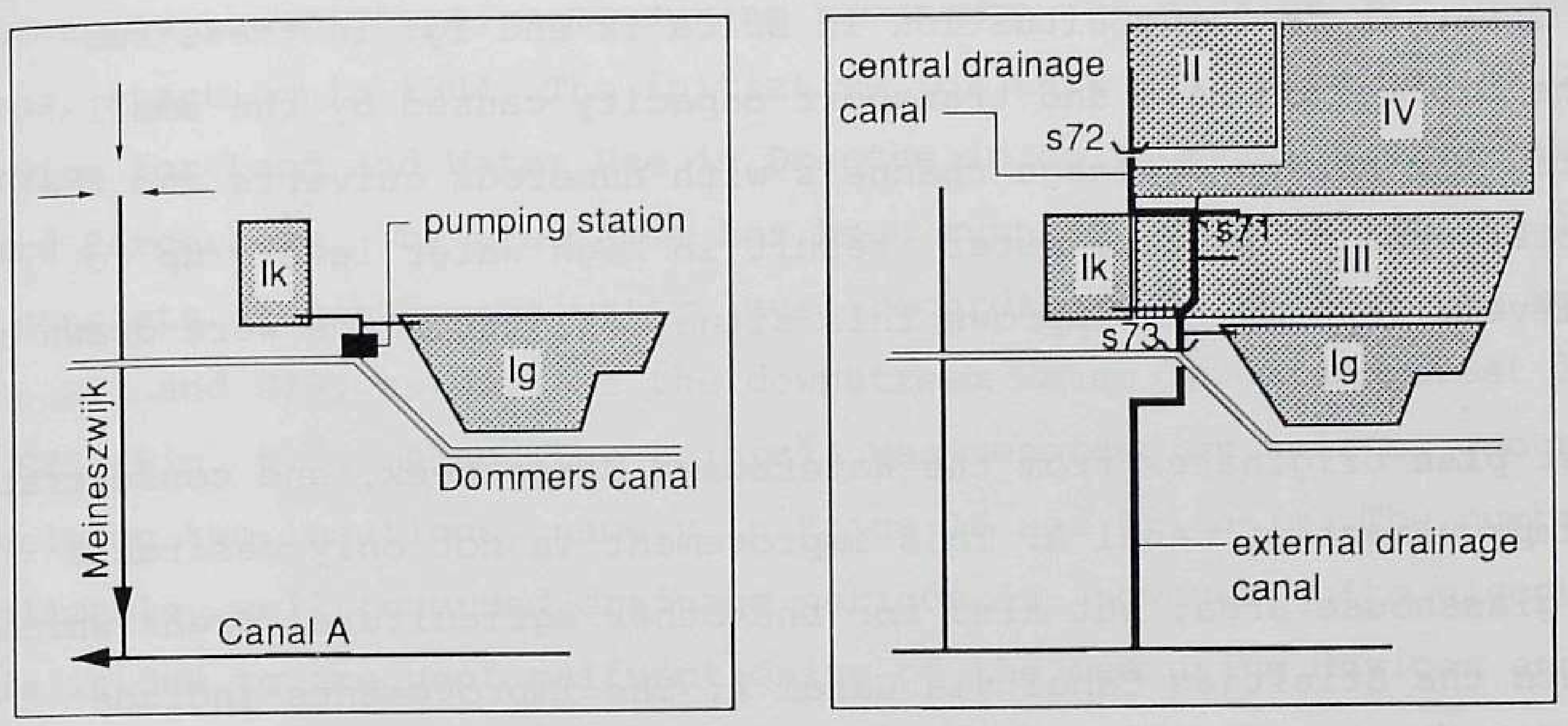


Fig.5.3. The development of the drainage situation in the glasshouse area.

was changed at that time by installing a pump discharging the water into the Dommers canal, the construction of a sub-surface drainage system and soil improving measures.

Because of excessively high groundwater levels the market gardeners complained about the drainage situation. The rose gardeners in particular suffered much damage caused by the high groundwater levels. The decision was therefore taken to change the drainage situation by abandoning the pump in favor of an inverted siphon under the Dommers Canal and to construct a large drainage canal to Channel A, the External Drainage Canal. At the same time, it was decided to provide a larger freeboard in the new area under development, Erica II, and to link the drainage of this area to the Central Drainage Canal to replace the existing link with the Meineszwijk. Erica III and IV would also be connected to the Central Drainage Canal. In this way drainage of the total horticultural area was assured via the External Drainage System into Canal A. The target level of the Central Drainage Canal is controlled by the automatic weir S73.

The installation of the External Drainage Canal in 1983 did not result in an improvement of the drainage situation in Erica Ik and Ig. This was caused by two factors. One is the limited transport capacity of Channel A. The resulting obstructed flow leads to higher water levels even upstream of weir S73. The target level of about NAP +11.20 m. of the Central Drainage Canal is therefore often exceeded. In extreme cases this leads to a reduced discharge from the glasshouse areas. The second factor is the bad internal drainage situation in Erica Ik and Ig. In these subareas the minimal storage and transport capacity caused by the small freeboard, the narrow drainage channels with numerous culverts and the small total area of surface water, result in high water levels up to ground level. In order to improve this situation, two plans were drawn up.

The first plan originates from the Waterboard Bargerbeek, and concentrates on the improvement of Canal A. This improvement is not only desirable for the glasshouse area, but also for the other agricultural areas which drain into the Stieltjes Canal via Canal A. The improvements include the replacement of a number of culverts in Canal A by larger culverts or bridges, and the reshaping of the profile of the channel by dredging out a large amount of silt together with a general deepening of the profile. In an attempt to obtain a subsidy for the costs, the plan was submitted to the Committee for Reallotment and Consolidation Works in the Drenthe-Groningen fen areas. The required approval was delayed, however, because a part of the silt was found to be polluted with phthalic esters, which are aromatic compounds used as base material for the production of plasticizers and polymers. Therefore the decision about an important part of the plan, the dredging of the silt, cannot be taken without referring to the regulations of the Chemical Waste Law. The Drenthe provincial authorities attempt to find a suitable solution.

The second plan, the Water Management Plan for the Erica Glasshouse Area comprises an improvement of the internal drainage situation in the glasshouse area. This plan was drawn up by a working group consisting of members provided by the Government Service for Land and Water Use, the Emmen Council, the Consulting Bureau for Horticulture, and the Trade Union for Horticulture Southeast Drenthe. The Water Board Bargerbeek takes care of the chairmanship. After the implementation of this improvement plan, control of the water management system in the glasshouse areas will be transferred from Emmen Council to the Water Board Bargerbeek. This plan is also awaiting approval for subsidy.

The essential steps in the drawing up of the improvement plans for Channel A and the internal situation in the glasshouse area, are the determination of the design discharge for glasshouse areas and the determination of the amount of excess water that is stored temporary in the soil profile. In order to provide a quantitative answer to questions concerning these steps, a measuring programme was carried out in these areas, starting in 1984. The initiative for this was taken by the Government Service for Land and Water Use in Drenthe in collaboration with the Water Board Bargerbeek. The programme has been running out from 1984 to 1988. It consists of continuous water level recordings upstream of the weirs S71, S72 and S73, as well as the downstream water level and crest level of S73 (Fig. 5.3). Also, the rainfall was recorded as well as groundwater levels on two locations, namely in Erica II and Erica Ig. The number of utilizable, well recorded drainage periods is, however quite discouraging. This is due to frequent malfunctioning of the measuring devices and datalogger and to organizational complications, in addition to sub-optimal positioning of the measuring devices, as appeared after evaluation of the data. Nevertheless the data recorded at the weirs S71, S72 and the downstream water level of S73 can very well be used for calibration and validation purposes. The phenomenon of the temporary storage of surplus water in the soil profile has been studied in an infiltration test in the autumn of 1985.

Objectives of the case

The objectives of this case can be formulated as follows:

To advice on the improvement of the drainage system in the present situation, anticipating on a future situation with a larger area of glasshouses. The advice should not only comprise the best solution, but preferably also an advice as to timing of the reconstruction works.

Before the variants can be developed and tested a model will have to be made of the drainage system and the model must be calibrated and validated.

5.3 Modelling the drainage situation

5.3.1 Schematization

The objective of a schematization of the model area is to translate the real situation into a simplified model situation where differences between calculated values of discharges and water levels and measured values are minimized, with the restriction that the degree of detail should not lead to unreasonable or unnecessarily high computation costs. Two

The drainage situation in Erica Ig

classes of data can be distinguished. The first class comprises data that can be determined relatively easy such as the place and the size of glasshouse complexes, the place and width of the watercourses and of the structures. The second class of data include data that cannot be determined accurately at reasonable costs such as the roughness coefficient, parameters of structures and parameters of the rainfall runoff relation applied. The values of this kind of data are determined in the calibration which is treated in the following section. In this section attention must be paid to the boundaries of the model area and the degree of detail; the rainfall runoff relations and the modelling of the ground storage.

The model boundaries and the degree of detail

Model boundaries are determined by the project area, provided that flows, either incoming or outgoing, are known or can be estimated, or water levels are known on the boundaries. The project area consists of the glasshouse areas and the main drainage channel, Canal A, up to the Stieltjes Canal. The boundary condition at the downstream end of Cannel A where the water level is controlled by a weir (S37), is determined by the water level of the Stieltjes Canal which is always below crest level of S37 and can be put at N.A.P. +9.80 m. Canal A contains water originating from five areas other than the glasshouse areas. These five sources discharge water either pumped up or passing over a weir. The first source concerns the runoff discharge of an industrial area of 43 ha. in addition to the constant discharge of about 0.12 $m^3.s^{-1}$ of cooling water. The second and third sources concern the runoff of catchment areas of 476 ha (via weir S69) and 1160 ha (via weir S70) respectively consisting merely of agricultural areas. The fourth source is related to an industrial area the runoff of which is stored and pumped into Channel A after purification. Crest levels, pump capacities and control mechanisms being known, the upstream water levels may well serve as model boundaries. The same holds true for the fifth source, the agricultural area Erica IV discharging into the Central Drainage Canal.

The number of nodes and thus the degree of detail of the schematization is determined first and foremost by the geometry of the network under

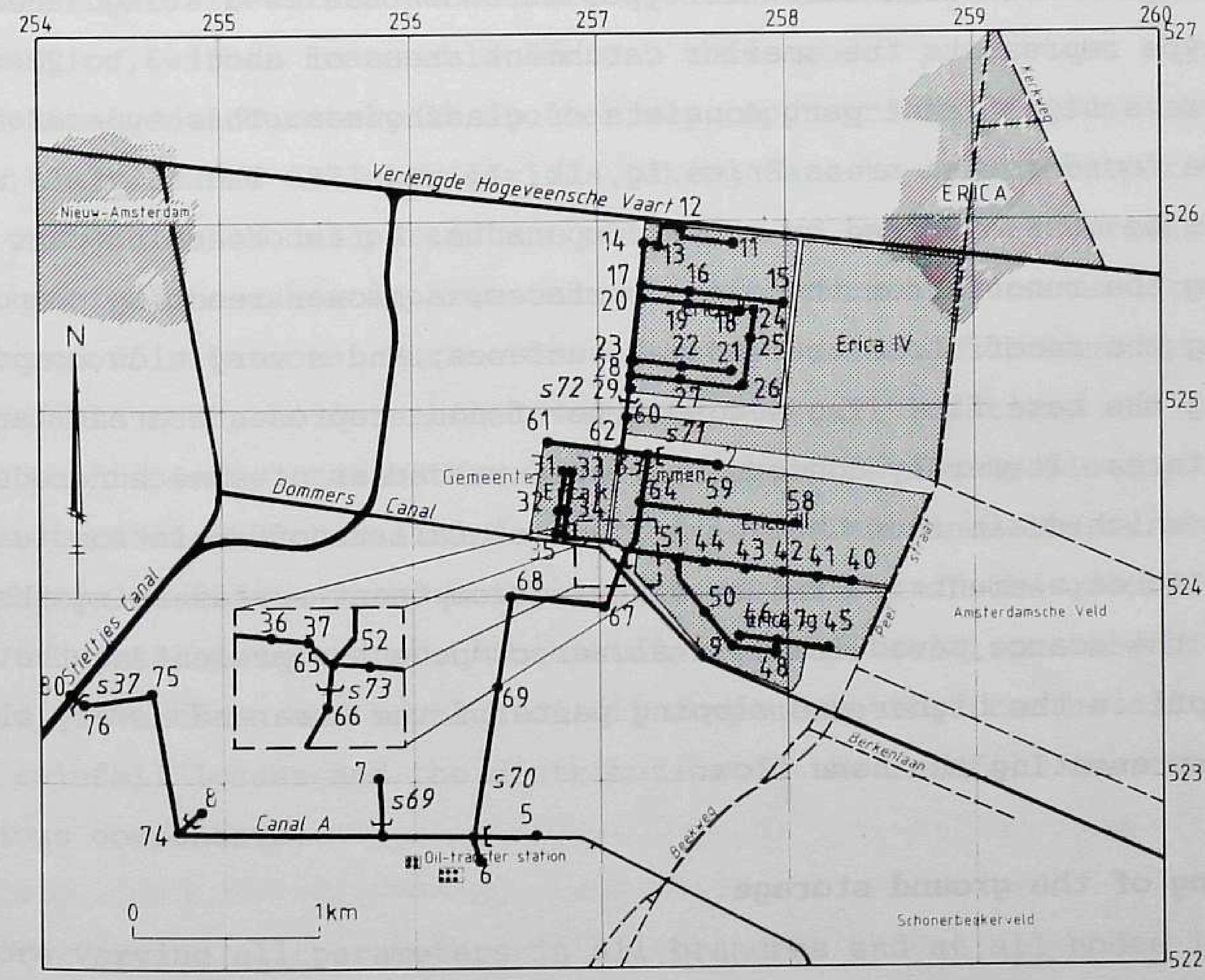


Fig.5.4. Schematization of the network of watercourses of the Erica glasshouse area with Canal A.

consideration. Nodes are placed at junctions of water courses, at locations where cross sections change significantly and at places where water is added to or withdrawn from the system. The necessary degree of detail of the schematization into nodes and branches is further determined by the degree of knowledge of the runoff mechanism, by the number of places where water levels or discharges are measured, by the possibilities of the model to cope with more structures in one branch and by the cost of the computations. The cost of the computations is determined by the cost of preparations (determining all input data) and the computation costs on the computer for one simulation. Studies into the necessary degree of refinement with respect to calculation results (Vreugdenhill, 1973; Velde, 1988) made it clear that for many problems a schematization into a system of main branches is sufficiently accurate if the secondary storage, which accounts for the open water storage in the secondary and tertiary channels, is taken into account.

The rainfall runoff relations

EWAS offers the possibility to vary the rainfall runoff relation for every node in the system. In this case two types of nodes can be distinguished. The first type represents the smaller catchment areas of about 3 to 20 hectare where a significant part consists of glasshouses. This type of nodes can be found in the areas Erica Ig, Ik, II and III. The rainfall runoff relation can be characterized by three components. A fast reacting part representing the runoff from the glass surfaces, a slower reacting component, representing the runoff from the unpaved surfaces; and a very slow component representing the base flow. The second type of node represents areas larger than 20 hectares. It merely concerns agricultural areas of some hundreds of hectares which drain into Channel A. The rainfall runoff relation is composed of three elements: a fastly reacting component representing the runoff from the scarce paved areas, a slower component representing the surface runoff in the higher and sloping parts of the area and a very slow component representing the base flow.

The modelling of the ground storage.

In the autumn of 1985 an infiltration test was carried out in the Erica glasshouse area. The objective was to determine the significance of the ground storage in this area. A watercourse in Erica II has been isolated

by dams and the water level was raised using a pump. The speed of the water level rise was seen to equal conditions during heavy rainfall. Groundwater levels were observed at a number of places at different distances from the water course. The results gave reason to conclude that the ground storage is significant in the glasshouse areas as soon as the level in the watercourses rises above drain level. Because of the very high drainage intensity in these areas and the rather high permeability of the soil the ground water level follows the water level with a time lag of less than two hours. More details can be found in Westra (1985). The upper limit for this supplementary storage in these glasshouse areas can be estimated at 5 % of the glasshouse area, equal to the estimated storage coefficient. Lacking more reliable data to account for this phenomenon, a flood plain at drain level was applied for the modelling of the watercourses in these areas.

5.3.2 Calibration

The calibration can be seen as a process where simulation results, in terms of water levels and discharges, are harmonized with measured data by varying, within reasonable limits, those model parameters which cannot be determined accurately in another way. Appropriate periods are chosen from the measured drainage periods taking care to save some periods for the validation. Model parameters which may be subject to variation in a calibration process are parameters of watercourses, structures and rainfall runoff relations. Parameters of watercourses which cannot, at reasonable cost, be determined accurately at all locations are the roughness coefficients, the bottom level and the size of the flood plains. Parameters of structures to be considered are entrance losses of culverts, discharge coefficients of weirs and the thickness of the groundfill in culverts. All parameters of the rainfall runoff relations are of importance, like the n and k value of all components that can be distinguished, the rainfall losses and the distribution of the rainfall over the various components.

Before varying all parameters in all branches and at all nodes in a system with 60 branches, the influence of these parameters on the results is determined. With respect to the characteristics of the drainage process the model area can roughly be divided into two parts. One part concerns the glasshouse areas Erica Ik, Ig and II, the second part concerns the Central Drainage Canal, Canal A and Erica III and IV.

The glasshouse areas

The drainage process in the glasshouse areas can be characterized as an initial storing of practically all the rain followed by a rather slow discharge of the water to the Central Drainage Canal. Two aspects account for this characterization. The first aspect involves the quickly reacting glass surfaces, causing the collected rain to reach the watercourse in a short time. The second aspect refers to the low transport capacity of the watercourses in these areas. This low transport capacity is due to a large number of small culverts. A large part of the head difference over the length of the channel in a drainage situation is utilized in passing these culverts. That storage is important in the drainage process can be illustrated with the velocities in the watercourses. Velocities in the channels in a drainage situation are 0.20 $m.s^{-1}$ on average, and never higher than 0.40 $m.s^{-1}$.

This characterization of the drainage process is important to explain the results with respect to the variation of the values of various model parameters such as the roughness coefficient, the bottom level of the watercourses, the groundfill and the entrance losses of culverts and the discharge coefficient of weirs.

The variation of the roughness coefficient between $k_m = 10$ and 30 $m^{1/3}.s^{-1}$ does not result, for example, in different water levels for node number 42 and 50. A variation of the bottom level by + or - 10 cm results in water levels that deviate no more than 1 cm in node number 40. Even the effects on the recession curve of the discharge from a particular sub area are negligible. The effect of a groundfill in the culverts of up to 10 cm is restricted to a maximum of 2 cm in upstream nodes like node number 40. The effects of a variation of the value for entrance losses of the culverts (between 0.06 and 1.50) is also limited to a few cm. In this case distinction can be made between up- and downstream nodes. A high value for the entrance losses results in higher waterlevels in upstream nodes and in lower waterlevels for downstream nodes. The latter can be explained by a smaller inflow of drain water from upstream. The effects of a variation of up to 40 % of the discharge coefficient of the weirs

are limited to a few cm in the first and the second nodes upstream of the weir. Further upstream the influence is diminished by the resistance of the culverts. The influence of the schematization of the ground storage by a flood plain is important. As can be expected the peak level as well as the recession curve are influenced by the presence of the flood plain. In the small area of Erica Ik with a rainfall of 25 mm in 3 hours (T=2) the peak water level in node number 31 decreased by 12 cm where the recession rate decreased from 5.5 to 2.8 cm per hour. The flood plain size applied is equal to the upper limit as discussed in Section 5.3.1 which gave satisfactory results. It has to be noted here that an error made in the determination of the storage area in the secondary drains influences the size of the flood plain to be applied.

The central drainage system

The central drainage system consists of the main watercourses from node number 61 up to node number 80 representing the Central and External Drainage Canal and Canal A. Designed to transport the water from the area to the Stieltjes Canal, its transport capacity is seriously hampered by 13 culverts, 10 of which are rather narrow compared to the dimensions of the channel. In the course of time these culverts have become rather deep lying, due to sedimentation of silt in the reaches between the culverts, the floor of which remains clear of sedimentation. Level differences between culvert bottom and channel bottom midway between culverts may increase to values of up to 50 cm.

This short description of the drainage process in the main drainage channels where the influence of the culverts is important, gives the impression of a channel which may have the right dimensions for the new situation, provided the culverts were to be reconstructed or replaced. The high resistance of the culverts is the reason that for the upstream part of the channel reach the same characterization holds as given for the glasshouse area. The resulting characterization for the whole channel reach is mixed and the effect of the variation of the various parameters is only partly different from the effects as described for the glasshouse areas. The roughness coefficient in the system considered here is varied between $k_m = 15$ and 55 $m^{1/3}.s^{-1}$. Its effect on water levels in nodes number 73 and 66 is limited to 4 cm. The effect of the variation of the bottom level by 10 cm is 2 cm. for node number 66 for example. The influence is important especially in reaches without culverts where the influence is translated into lower velocities. Although it was considered a most unlikely occurence, the culverts in the model have been given a groundfill of 10 cm, which resulted in water level differences of about 4 cm. This can be explained by realizing that the groundfill corresponds to a diminishing of the wetted area in the culverts of about 6%, and that the result corresponds to a diminishing of 8% of the total head difference of 47 cm over the channel reach from node number 66 to node number 76. The effects of the variation of the entrance losses of the weirs, limited to the two most likely values of 0.25 to 0.44, were negligible. The effect of the variation of the discharge coefficient of weirs was also negligible because the weirs in this part of the drainage system are automated devices reacting to the upstream water level.

The calibrations

The calibration procedure itself is hampered by the fact that data concerning the land uses and especially regarding the area of glasshouses in the years up to 1988 were not available. They could only be derived from maps which were altered several times, neglecting to provide information about the dates of the changes. Furthermore, the calibration has necessarily to be limited to the upstream water levels of the weirs S71 and S72 and the downstream water level of S73, the latter providing some information about the functioning of Channel A. No additional information is available apart from some 'soft' information, like the location of the places where serious difficulties were encountered with water rising up to inundation levels. Other disturbing factors for the calibration procedure are that in the most interesting drainage periods the control mechanism of the automatical weirs is set to manual control without any information with respect to the time of this action nor to the kind of control applied by hand. At the same time often additional pumping capacities have been applied or dikes have been demolished to avoid serious damage for the gardeners.

Considering the above, the calibration procedure can be restricted to the determination of the parameter values of the components of the rainfall runoff relations of the various catchment areas. The following reasoning will provide a further condensing to the slower components of the rainfall runoff relations.

The rainfall runoff relation of small nodes consists of:

. a part reacting quickly, representing the runoff from the glass surfaces. Applied in the Nash cascade, the choice in this case is a combination of n=2 with k=0.002 days based on experiences with glasshouse areas in the Westland. A variation of these parameters so that the peak discharge occurs either at the end of the rain period or with a delay of 15 minutes did not result in different water levels or discharges. A wider range of variation is not considered to be realistic. Rainfall losses are estimated at 1 mm in those cases where no rainfall occurred in the period of the preceding 24 hours.

. a component reacting slowly, representing the runoff from the unpaved surfaces. In this kind of nodes these consist of well drained gardens. The choice of the Nash cascade parameters does not affect the height nor the time of peak water levels or peak discharges. It does affect the recession curve. The values applied (n=3, k=0.1 days) are determined on the basis of calibrations of Erica II. Rainfall losses are estimated on the basis of the discharge of Erica IV, a 100% agricultural area.

. a very slow component representing the base flow or the seepage from higher grounds in the east of the area, and the relatively high level of the Verlengde Hoogeveense Vaart. Values of this constant discharge were derived from Erica IV.

The rainfall runoff relation of the large nodes is composed of the following three elements:

. a component of fast reaction, representing the runoff from the scanty paved areas like small industrial areas and roads. The measurements available were insufficient to determine the Nash parameters. These had to be estimated (n=2, k=0.08). Rainfall losses are taken to be the same as for the glasshouses.

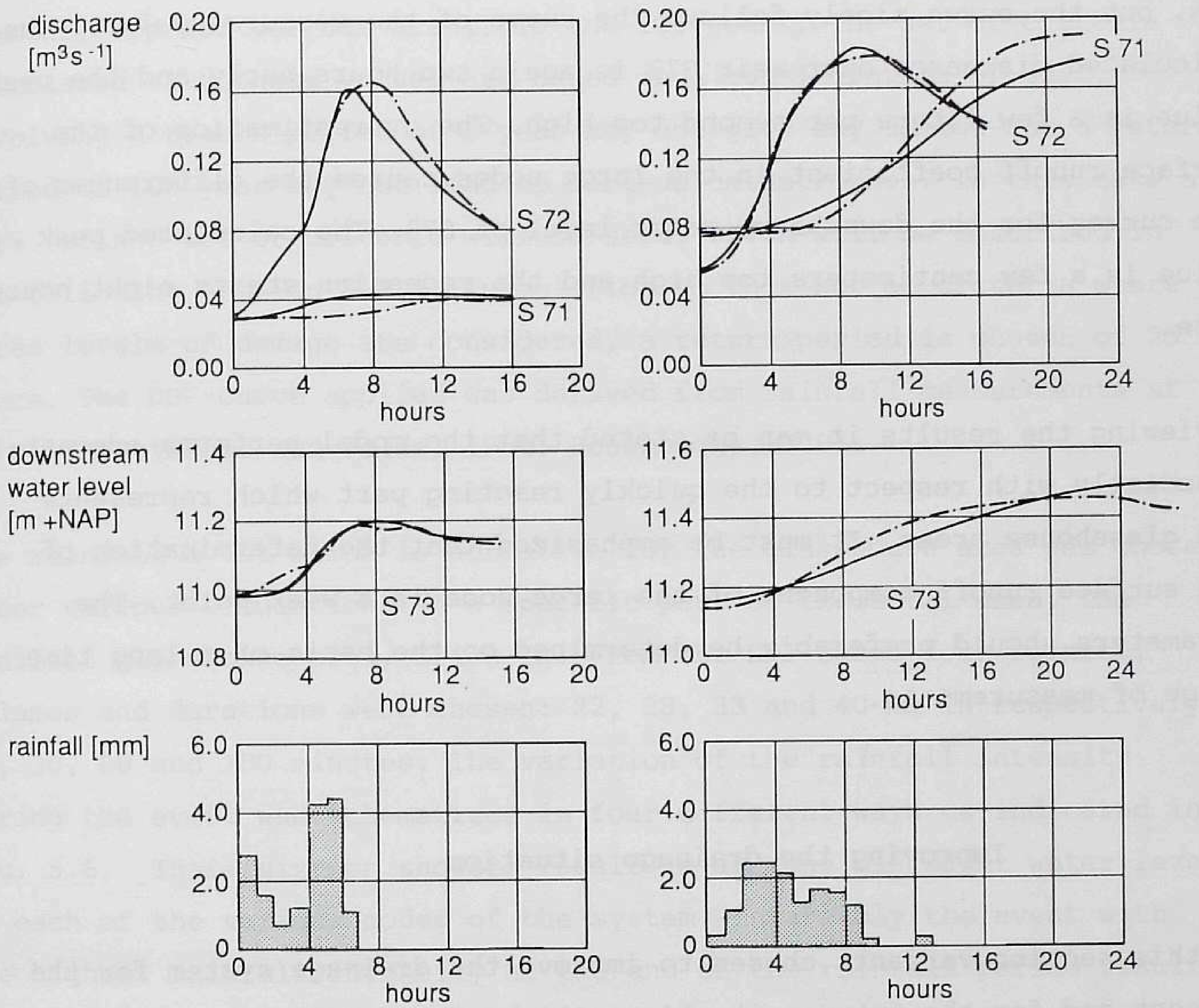
. a slower component representing the surface runoff or interflow component in the higher and sloping parts of the area. The data give reason to believe in the existence of this component. It could be distinguished after a period of heavy rainfall with high intensity, but also in a very wet period with less rainfall. The phenomenon has also been observed after periods of frost. In both cases the infiltration capacity is not sufficient and a part of the rainwater is discharged over the surface into the watercourses. Parameter values yielding satisfactory results are n=4 and k=0.2.

- a very slow component representing the seepage flow originating from higher grounds and the slow groundwater discharge especially in the areas upstream of nodes number 2, 5 and 7. This component is simulated by a constant drain discharge, the value of which is derived from the discharge of Erica IV. This is an acceptable approach as long as the simulated period is not longer than 24 hours.

For this type of nodes it is important to determine what part of the net rainfall is discharged by the third component and what part by the second component. This largely depends on the time of the year, the rainfall intensity, the total amount of rainfall and the frost conditions in the preceding period as well as during the rainfall. The available data does not contain a sufficient number of rainfall events over a number of years to detect a relation between these variables. This distribution problem can be seen as the most delicate and also as the weakest point of the simulations. The approach followed here consists of an interpretation of the discharge of Erica IV to determine the base flow discharge and to determine whether a surface runoff component exists. If this component exists the percentage of the rainfall for this component is estimated and, if possible, adjusted after the first run.

5.3.3 Validation

The validation of the model has been carried out for two periods. The first period concerns a rainfall event of 16 mm in 3 hours in the month of October 1987. Based on the discharge over S71 the base flow is estimated at 2.0 $mm.day^{-1}$. The data furthermore showed no reaction of the surface runoff component which can be explained by a rather dry preceding period. The results of this validation run are given in Fig. 5.5. It is evident from this figure that the model yields satisfactory results with respect to the discharge of weir S72 and S71. The measured discharge of S71 reacts rather slowly: its initial discharge of 0.032 $m^3.s^{-1}$ increases to 0.040 $m^3.s^{-1}$ in 16 hours whilst the model reaches this value in 6 hours. As to the discharge of Erica II over weir S72 it can be stated that the height of the model peak discharge is correct but



validation results of the model for a rainfall event of 16mm in 6 hours

validation results of the model for a rainfall event of 15mm in 13 hours

Fig. 5.5. Validation results of the model for two rainfall events

is reached 2 hours early. The downstream water level of weir S73 is simulated adequately. The curve of the calculated levels is smoother because the model weir does not react as abruptly as the real weir.

The second period concerns a rainfall event of 15 mm in 13 hours in the month of November 1987. The base flow is estimated at 5.0 $mm.day^{-1}$. The fact that this is higher than the foregoing period can be explained by the time of the year. The data furthermore showed a reaction of the surface runoff component. Based on experience from the calibrations, net rainfall resulting in surface runoff or interflow is estimated at 40% of the total amount. The results of this run are given in the second set of graphs in Fig. 5.5.

This figure shows that the model performs well with respect to the discharges of the weirs and the downstream water level of weir S73. In this second run the discharge over weir S71 is overestimated by about 10%, but the curve nicely follows the curve of the measured values. The calculated discharge over weir S72 is again two hours early and the peak value is a few liters per second too high. The overestimation of the surface runoff coefficient in the large nodes causes the differences of the curves for the downstream water level of S73. The calculated peak value is a few centimeters too high and the recession starts eight hours late.

Reviewing the results it can be stated that the model performs adequately, especially with respect to the quickly reacting part which represents the glasshouse areas. It must be emphasized that the determination of the surface runoff component of the large node is a weak point. The parameters should preferably be determined on the basis of a long time range of measurements.

5.4 Improving the drainage situation

In this section variants chosen to improve the drainage system for the present and for the future situation will be developed and evaluated. Furthermore a time schedule will be presented as to the timing of the reconstruction works. In the following sub-section 5.4.1 attention will be paid to the choice of the design rainfall and in sub-section 5.4.2 the design criteria applied will be described. In sub-section 5.4.3 the variants will be introduced and evaluated.

5.4.1 Design rainfall

In this case the design rainfall is chosen from a DDF-curve by exploring the whole curve with several chosen variations of the rainfall intensity during the event, in search of the particular event that results in the highest water levels. Since two different types of areas are involved, namely the glasshouse areas on the one hand and the stretch of Canal A where the runoff from the agricultural areas may be normative on the other hand, two different rainfall events will be considered. Both events will be used as design rainfalls in the evaluation of the variants. The selection of the return period of the DDF-curve is linked to the set of criteria used. As outlined in Section 3.2.4. the most commonly applied periods are 1, 10 and 100 years. The set of criteria associated with these values are based on a consideration of cost, involving a return period of 1 year may not give any damage and a return period of 100 year may not lead to serious catastrophes. In this case a more refined set of criteria will be used, which will be described in the following Section 5.4.2. In relation to this set of criteria where three levels of damage are considered, a return period is chosen of 25 years. The DDF-curve applied was derived from rainfall measurements at De Bilt (Buishand and Velds, 1980; Bouwkneght,1988).

The rainfall event which is normative for the glasshouse ares was chosen after various simulations on a specific part of the total area, the smallest glasshouse area, Erica Ik. From the DDF-curve four rainfall volumes and durations were chosen: 22, 28, 33 and 40 mm in respectively 15, 30, 60 and 180 minutes. The variation of the rainfall intensity during the event was schematized in four different ways as indicated in Fig. 5.6. These sixteen showers result in as many different water levels in each of the various nodes of the system. Invariably the event with the highest rainfall intensity at the end of the rainfall period results in the highest water levels. This can be explained by realizing that in those cases the available storage is consumed at the beginning of the event leaving smaller storage possibilities for that part of the event with the highest intensities when the storage is needed most.

Interesting differences between the patterns of change of the various water levels in the different nodes of the system are determined by the place in the system (at the end of a watercourse or at the outlet) or by

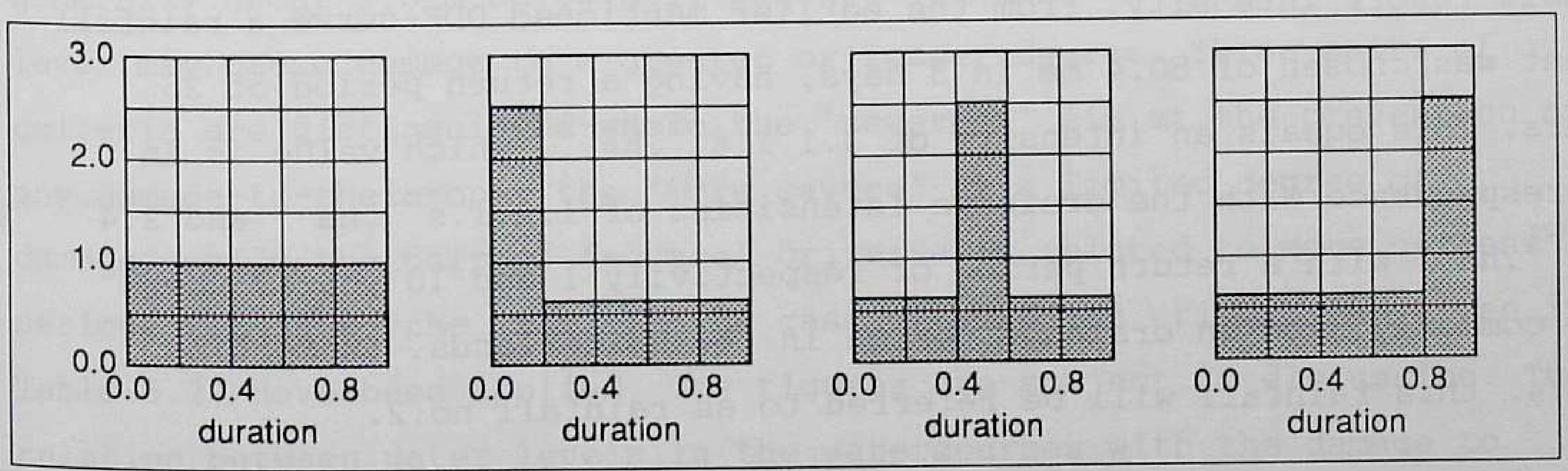


Fig. 5.6. Four different shapes of the rainfall events.

the different directly available storage capacities of the nodes. In Fig. 5.7. the course of the water level in the nodes numbered 33 and 35 is given for the four different rainfall events, each with its highest intensity at the end of the shower. The simulations described above resulted in a design rainfall of 33 mm in 60 min for the glasshouse areas with its highest rainfall intensity at the end of the rainfall period. In Section 5.4.4. this rainfall will be referred to as rainfall no.1.

A similar procedure might also have been carried out for the main channel reach from node number 61 to node number 80. This has been rejected because of the great uncertainties in the runoff behaviour of the large agricultural areas. Instead it was assumed that the normal event has a duration of several days with a uniformly distibuted rainfall intensity. In this way, a steady state can be reached where rainfall intensity equals runoff intensity. From the earlier mentioned DDF-curve a rainfall event was chosen of 80.4 mm in 3 days, having a return period of 25 years. This equals an intensity of 3.1 $l.s^{-1}.ha^{-1}$, which value is in correspondence with the drainage intensities of 1.2 $l.s^{-1}.ha^{-1}$ and 2.4 $l.s^{-1}.ha^{-1}$ with a return period of respectivily 1 and 10 years: these are commonly used in drainage design in the Netherlands. In Section 5.4.4. this rainfall will be referred to as rainfall no.2.

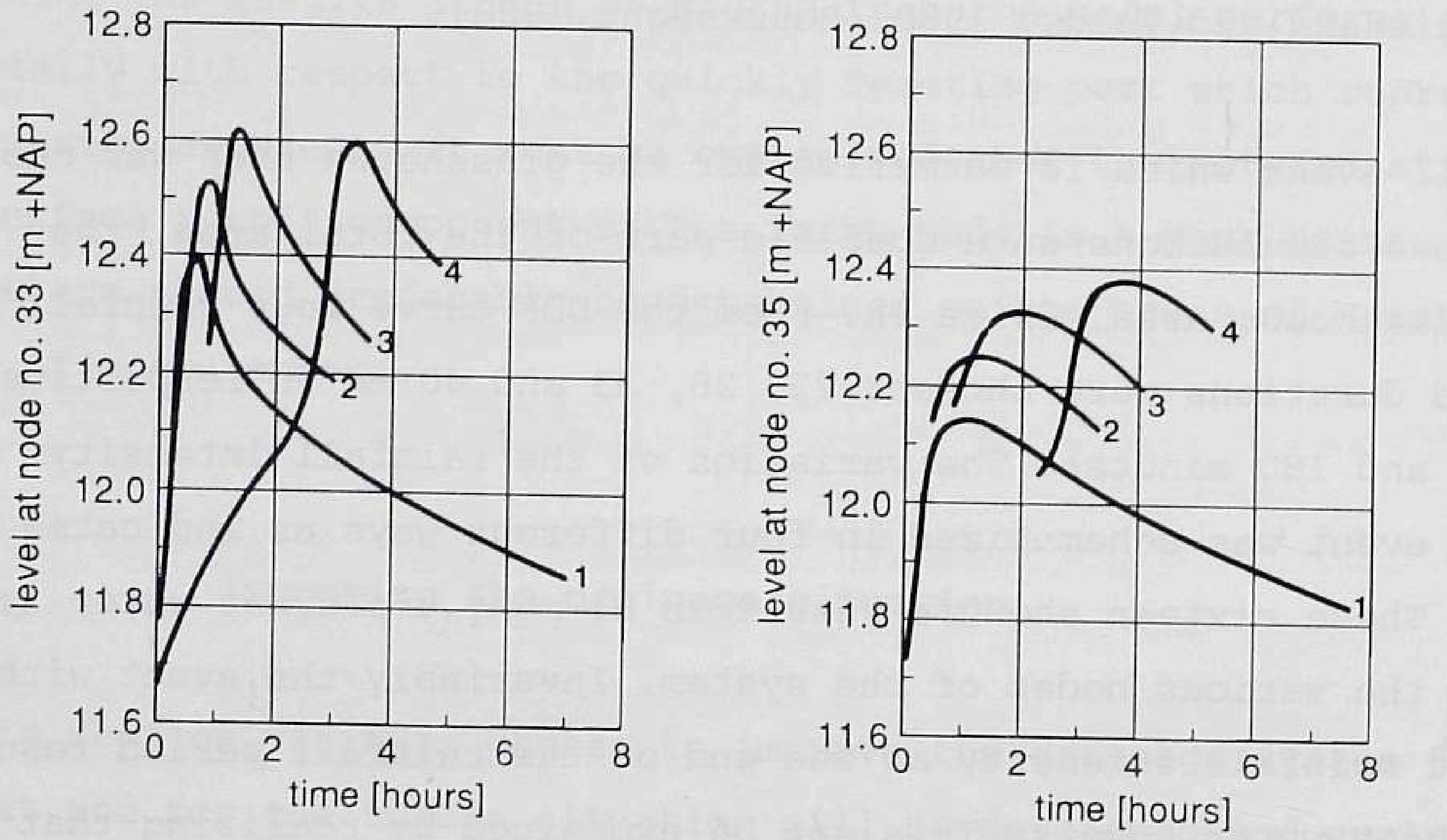


Fig. 5.7. The water level changes in nodes 33 and 35 for the four rainfall events, each with its highest intensity at the end of the shower.

5.4.2 Design criteria

The re-design is aimed at the prevention of high water levels in the glasshouse areas. Design criteria have to be established for the evaluation of the variants. High water levels may cause high groundwater levels in the glasshouses which may damage the crops. Both the height and the duration of a certain high water level are important factors that determine the damage. This leads to pairs of design criteria and a application of EWAS as suggested in Section 4.1 and indicated in Table 4.2. In this case three pairs of design criteria were applied in the glasshouse areas and two pairs in the nodes of the Internal and External Drainage Canal and Canal A. These criteria as formulated below, may all be subject to discussion. Because criteria are not available, the approach followed here is, that rainfall events which occur often must satisfy the 'severest' criteria while the rarer events must meet the 'more severe' or the 'severe' criteria only. This idea is illustrated in Table 5.1.

Table 5.1. The relation between the criteria and the occurrence of the rainfall events.

occurrence	Criteria		
	severe	more severe	severest
several times a year	+ +	+ +	+ +
once in 1 to 5 years	+ +	+ +	- -
once in 10 to 25 years	+ +	- -	- -
once in 50 to 100 years	- -	- -	- -

Glasshouse areas

The criteria in the glasshouse areas are related to the drain depth, generally about 1.0 m below soil surface. Groundwater levels above this level may cause damage to a greater or lesser degree. Three pairs of criteria are distinguished where the 'severest' aim at the prevention of any damage to the crops, the 'more severe' at a limited degree of damage, while the pair of 'severe' criteria is related to more or less serious damage to the crop. In the case at hand the criteria as given in Table 5.2. have been applied. The figures are subject to discussion. The relation between water levels in the watercourses with the damage to crops in the glasshouses depends, amongst others, on the sensitivity of

Table 5.2. Criteria as applied in the glasshouse areas

	criterion	level with respect to soil surface [m.]	duration [hours]
severe	steady	-0.20	
	non steady	-0.50	6.0
more severe	steady	-0.75	
	non steady	-0.50	6.0
severest	steady	-1.00	
	non steady	-0.75	6.0

the crop to too wet conditions, the permeability of the soil, the drainage intensity, the resistance to infiltration and the development stage.

The main drainage canal

The criteria for the Central, Internal and External Drainage Canals and for Canal A are formulated as suggested by Smedema and Rycroft (1983). They provide a freeboard of 0.50 m for the larger canals. This freeboard is adopted here for the 'severe'steady state criterion. The corresponding non steady state criterion of 0.75 m combined with a duration of 24 hours was chosen subjectively as a first approach to such criteria. For the 'more severe' criteria these values are 0.80 m for the steady state, and 1.0 m with 24 hours duration for the non-steady state criterion.

5.4.3 The evaluation of five variants

In this section variants will be discussed that are considered to solve the water clogs in the glasshouse areas. Not all variants that have been taken into account will be discussed here. One design philosophy has been to create more storage capacity in the direct neighborhood of the glasshouses, because, generally speaking, there is insufficient time during the rainfall event to transport the water. Calculations based on this have been carried out and it proved to be a good solution. The

variant was rejected however, because the optimal storage area could not be realized at reasonable cost. Another variant where both the transport capacity and the storage capacity of the existing system of watercourses were increased, was rejected for the same reason. A third variant, originating from the Erica drainage working group has been evaluated. It proved to be a good approach although it needed some modifications. This modified plan, presented here as variant 2 and described in the following, satisfies the most part of the criteria at a minimum of cost. In the following, five variants will be discussed. Variant 1 presents the existing situation under design conditions. The variants 3, 4 and 5 present a decisive answer concerning the planning of the works if these are carried out in phases in the coming years. This includes a directive as to the part of the works the execution of which should be given priority.

Variant 1, the present situation under design conditions

This variant (Fig.5.8) concerns the present situation under design conditions. Design conditions refer to the expected situation in the near future with a glasshouse area increased upto 80 % in the areas Erica Ik, Ig, II and III, and the secondary drains, separating the glasshouses, filled up in the areas Erica Ik and Ig. The words 'expected situation' mean that no measures have been taken to increase the storage capacity and/or the transport capacity of the watercourses. The results, given in Table 5.3. give reason the the following remarks:

The first remark concerns the total number of minuses for both rainfall events. It makes clear that in such a situation many of the criteria are not met. In both glasshouse areas Erica Ik and Ig serious damage may be expected at each of the two rainfall events. In many places of these areas neither the steady state criteria nor the non steady state criteria are met. In the new glasshouse area Erica III all criteria are met. The Erica II and III areas are well designed and no damage is expected to occur. Especially for rainfall no.2., flooding and serious damage may be expected in the main drainage channel between node number 67 and 71. The results show clearly that measures are desirable. Secondly the score table makes clear that the rainfall no.1, which is of short duration and high intensity results in many minuses in the glasshouse areas but is of no importance to the Main Drainage system from nodes numbered 60 to 76. The rainfall event no.2 gives very high water levels in the main drainage system and causes a hampered drainage of the glasshouse areas. So much so, that the weir S70 is completely submerged and in some cases a return flow from the External Drainage Canal discharges water to the glasshouse areas, where the open water surface is used to store the excess water from the agricultural areas especially from behind node numbers 5 and 7. Water levels are increased so much that even the culverts which normally discharge freely into the Central Drainage Canal are submerged. It is this hampering of the discharge which causes the high water levels in the glasshouse areas more than the rainfall on the glasshouses itself. The financial results indicate a yearly amount for maintenance of about kDfl 60.0 and a net present value of the existing system of kDfl 5272.0

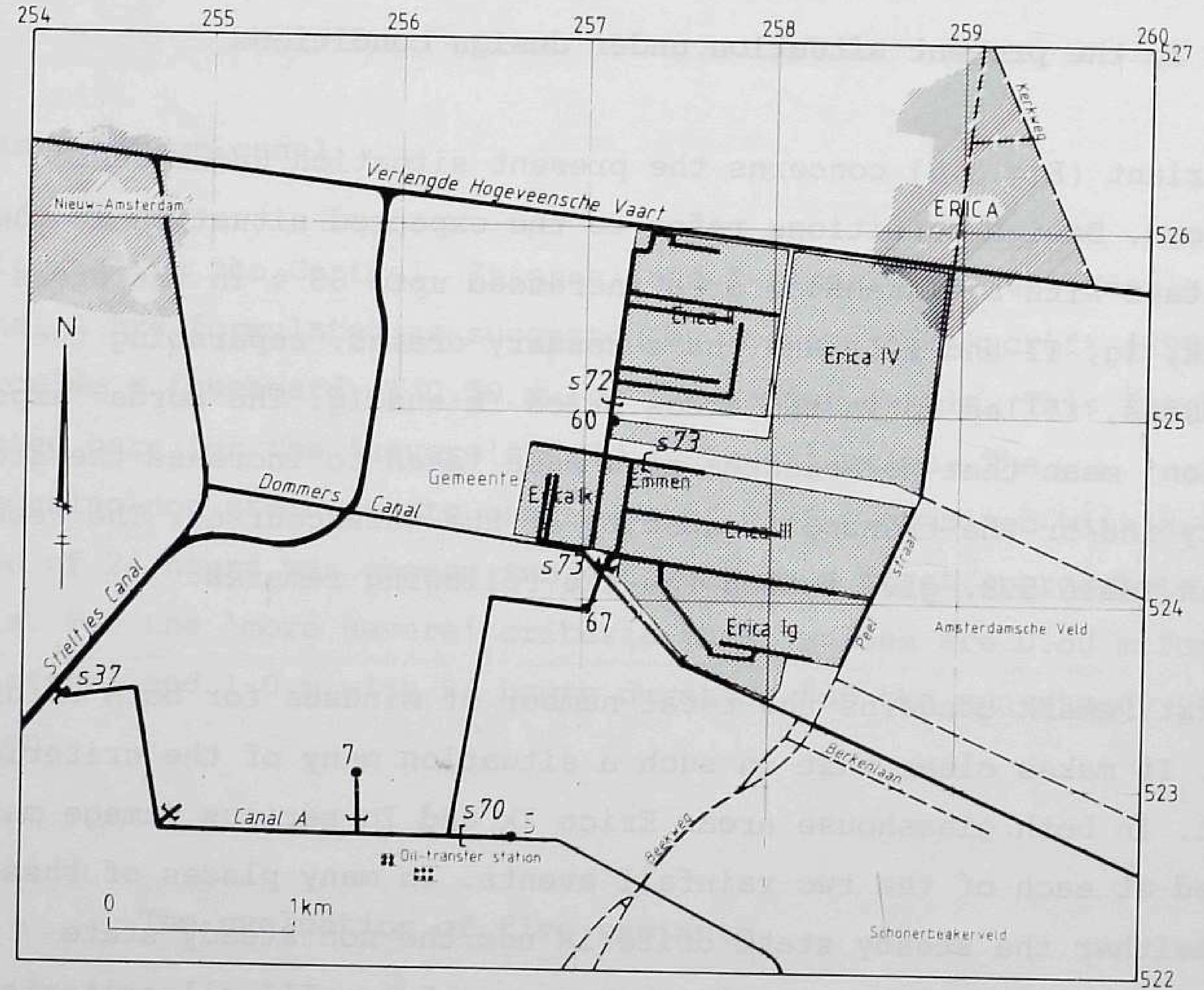


Fig.5.8. Schematization of the network of watercourses of the Erica glasshouse area with Canal A in the present situation under design conditions with increased glasshouse areas.

Table 5.3. The evaluation of Variant 1

node number	Criteria rainfall no.1						Criteria rainfall no. 2						
	severe		more severe		severest		severe		more severe		severest		
2	+	+	+	+	***	***	-	-	-	-	***	***	Large agricultural areas
5	+	+	+	+	***	***	-	-	-	-	***	***	
7	+	+	+	+	***	***	+	+	-	-	***	***	
8	+	+	***	***	***	***	+	+	***	***	***	***	
11	+	+	-	+	-	+	+	+	+	+	+	-	Erica II
12	+	+	-	+	-	+	+	+	+	+	+	-	
13	+	+	+	+	-	+	+	+	+	+	+	-	
14	+	+	+	+	-	+	+	+	+	+	+	-	
15	+	+	+	+	+	+	+	+	+	+	+	-	
16	+	+	+	+	+	+	+	+	+	+	+	-	
17	+	+	+	+	+	+	+	+	+	+	+	-	
18	+	+	-	+	-	-	+	+	+	-	-	-	
19	+	+	-	+	-	-	+	+	+	-	-	-	
20	+	+	+	+	+	+	+	+	+	+	+		
21	+	+	-	+	-	-	+	+	-	-	-	-	
22	+	+	-	+	-	-	+	+	+	-	-	-	
23	+	+	+	+	+	+	+	+	+	+	+	-	
24	+	+	-	+	-	-	+	+	-	-	-	-	
25	+	+	-	-	-	-	+	+	-	-	-	-	
26	+	+	-	-	-	-	+	+	-	-	-	-	
27	+	+	+	+	-	-	+	+	-	-	-	-	
28	+	+	+	+	+	+	+	+	+	+	+	-	
29	+	+	+	+	+	+	+	+	+	+	+	-	
31	-	-	-	-	-	-	-	-	-	-	-	-	Erica Ik
32	+	-	-	-	-	-	-	-	-	-	-	-	
33	-	-	-	-	-	-	-	-	-	-	-	-	
34	+	-	-	-	-	-	-	-	-	-	-	-	
35	+	-	-	-	-	-	-	-	-	-	-	-	
36	+	+	+	-	-	-	-	-	-	-	-	-	
37	+	+	+	-	-	-	-	-	-	-	-	-	
40	-	-	-	-	-	-	-	-	-	-	-	-	Erica Ig
41	-	-	-	-	-	-	-	-	-	-	-	-	
42	-	-	-	-	-	-	-	-	-	-	-	-	
43	-	-	-	-	-	-	-	-	-	-	-	-	
44	+	+	-	-	-	-	-	-	-	-	-	-	
45	-	-	-	-	-	-	+	-	-	-	-	-	
46	-	-	-	-	-	-	-	-	-	-	-	-	
47	+	-	-	-	-	-	+	-	-	-	-	-	
48	-	-	-	-	-	-	-	-	-	-	-	-	
49	-	-	-	-	-	-	-	-	-	-	-	-	
50	-	-	-	-	-	-	-	-	-	-	-	-	
51	-	+	-	-	-	-	+	-	-	-	-	-	
52	+	+	-	-	-	-	+	-	-	-	-	-	
58	+	+	+	+	+	+	+	+	+	-	-	-	Erica III
59	+	+	+	+	+	+	+	-	-	-	-	-	
60	+	+	+	+	***	***	+	+	-	-	***	***	Central Drainage Canal
61	+	+	+	+	***	***	+	+	+	-	***	***	
62	+	+	+	+	***	***	+	+	+	-	***	***	
63	+	+	+	+	***	***	+	-	-	-	***	***	
64	+	+	+	+	***	***	+	-	-	-	***	***	
65	+	+	+	+	***	***	+	-	-	-	***	***	
66	+	+	+	+	***	***	+	-	-	-	***	***	External Drainage Canal
67	+	+	+	+	***	***	-	-	-	-	***	***	
68	+	+	+	+	***	***	-	-	-	-	***	***	
69	+	+	+	+	***	***	-	-	-	-	***	***	
71	+	+	+	+	***	***	-	-	-	-	***	***	Canal A
72	+	+	+	+	***	***	+	+	-	-	***	***	
73	+	+	+	+	***	***	+	+	+	+	***	***	
74	+	+	+	+	***	***	+	-	-	-	***	***	
75	+	+	+	+	***	***	+	-	-	-	***	***	
76	+	+	+	+	***	***	+	+	+	-	***	***	

Investment costs	[kDfl]	000.0
Maintenance costs	[kDfl]	60.0
Net present value of the costs	[kDfl]	5272.0

Variant 2, the future situation with completely modified drainage system

This variant concerns the same situation as variant 1, as far as the glasshouse areas and the filled up secondary drains are concerned. The drainage system, in both the glasshouse areas and the main drainage canals has been changed according to the plans of the Erica Drainage Working Group. The plans, as indicated in Fig. 5.9., implicate the modification of some existing tertiary drains (branch 30 to 61 and 38 to 62) and the creation of a number of new drains (branch 55 to 53 and 47 to 58). The runoff from the glasshouses is now diverted through these new ditches which provide the necessary storage and transport capacity. The drain discharge originating from the sub-surface drainage system of the glasshouses continues to be conducted through the existing system of watercourses. The main drainage canal from node number 71 to 76, Channel A, has been modified in this plan. All of the culverts except one have been replaced by concrete bridges. In a earlier variant which is not presented here, the culverts had been replaced by culverts of larger dimensions.

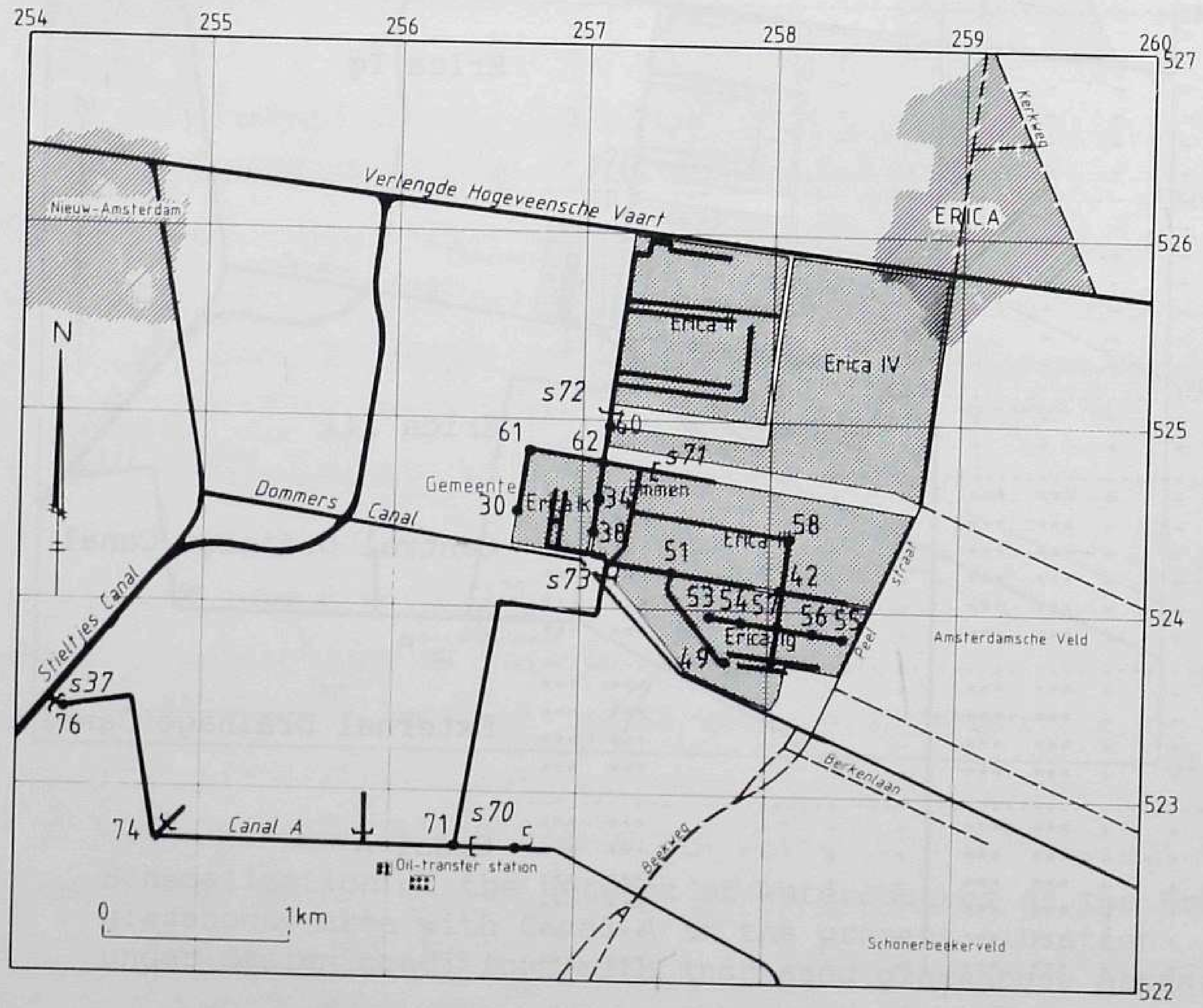


Fig.5.9. Schematization of the network of watercourses of the Erica glasshouse area with Canal A in the future situation with completely modified drainage system according to variant 2.

Table 5.4. The evaluation of Variant 2

node number	Criteria rainfall no.1						Criteria rainfall no. 2						
	severe		more severe		severest		severe		more severe		severest		
2	+	+	+	+	***	***	+	+	-	-	***	***	Large agricultural areas
5	+	+	+	+	***	***	+	+	-	-	***	***	
7	+	+	+	+	***	***	+	+	-	-	***	***	
8	+	+	***	***	***	***	+	+	***	***	***	***	
11	+	+	-	+	-	+	+	+	+	+	+	+	Erica II
12	+	+	-	+	-	+	+	+	+	+	+	+	
13	+	+	+	+	-	+	+	+	+	+	+	+	
14	+	+	+	+	-	+	+	+	+	+	+	+	
15	+	+	+	+	+	+	+	+	+	+	+	+	
16	+	+	+	+	+	+	+	+	+	+	+	+	
17	+	+	+	+	+	+	+	+	+	+	+	+	
18	+	+	-	+	-	-	+	+	+	+	+	-	
19	+	+	-	+	-	-	+	+	+	+	+	-	
20	+	+	+	+	+	+	+	+	+	+	+	+	
21	+	+	-	-	-	-	+	+	+	+	+	+	
22	+	+	-	+	-	-	+	+	+	+	+	+	
23	+	+	+	+	+	+	+	+	+	+	+	+	
24	+	+	-	+	-	-	+	+	+	+	+	-	
25	+	+	-	-	-	-	+	+	+	-	-	-	
26	+	+	-	-	-	-	+	+	+	-	-	-	
27	+	+	+	+	-	-	+	+	+	+	+	-	
28	+	+	+	+	+	+	+	+	+	+	+	+	
29	+	+	+	+	+	+	+	+	+	+	+	+	
30	+	+	+	+	+	+	+	+	+	+	+	-	Erica Ik
38	+	+	+	+	-	+	+	+	+	+	+	-	
39	+	+	+	+	+	+	+	+	+	+	+	-	
49	+	+	-	+	-	+	+	+	+	+	+	+	Erica Ig
50	+	+	-	+	-	+	+	+	+	+	+	+	
51	+	+	-	+	-	+	+	+	+	+	+	+	
52	+	+	+	+	-	+	+	+	+	+	+	+	
48	+	+	-	+	-	+	+	+	+	+	+	+	
47	+	+	-	+	-	+	+	+	+	+	+	+	
53	+	+	-	+	-	+	+	+	+	+	+	+	
54	+	+	-	+	-	+	+	+	+	+	+	+	
55	+	+	-	+	-	+	+	+	+	+	+	+	
56	+	+	-	+	-	+	+	+	+	+	+	+	
57	+	+	-	+	-	+	+	+	+	+	+	+	
42	+	+	+	+	+	+	+	+	+	+	+	+	
58	+	+	+	+	+	+	+	+	+	+	+	+	Erica III
59	+	+	+	+	+	+	+	+	+	+	+	-	
60	+	+	+	+	***	***	+	+	+	+	***	***	Central Drainage Canal
61	+	+	+	+	***	***	+	+	+	+	***	***	
62	+	+	+	+	***	***	+	+	+	+	***	***	
63	+	+	+	+	***	***	+	+	+	+	***	***	
64	+	+	+	+	***	***	+	+	+	+	***	***	
65	+	+	+	+	***	***	+	+	+	+	***	***	
66	+	+	+	+	***	***	+	+	+	+	***	***	External Drainage Canal
67	+	+	+	+	***	***	+	+	+	+	***	***	
68	+	+	+	+	***	***	+	+	+	+	***	***	
69	+	+	+	+	***	***	+	+	+	+	***	***	
71	+	+	+	+	***	***	+	+	+	+	***	***	Canal A
72	+	+	+	+	***	***	+	+	+	+	***	***	
73	+	+	+	+	***	***	+	+	+	+	***	***	
74	+	+	+	+	***	***	+	-	-	-	***	***	
75	+	+	+	+	***	***	+	+	+	+	***	***	
76	+	+	+	+	***	***	+	-	-	-	***	***	

Investment costs	[kDfl]	868.0
Maintenance costs	[kDfl]	60.0
Net present value of the costs	[kDfl]	5925.0

It appeared that this measure was not sufficient in terms of the criteria. In the variant at hand, furthermore, Canal A is reshaped. The reshaping is done by deepening the bottom level of the channel over its full length. It appears difficult to realize this deepening especially at the end of the channel. As has been mentioned in Section 5.2., the silt of the bottom is polluted and approval for dredging works has been delayed.

The results, given in Table 5.4. illustrate that the drainage system performs according to the design criteria. Improvement is obtained in all areas where serious damage is expected: Erica Ik and Ig and along the main drainage canal from node number 60 up to 76. Only at the nodes numbered 74 and 76 the non steady state criterion of the severe group is not met.

The financial evaluation shows an amount of money for maintenance equal to that of variant 2. This result is due to the fact that the frequency of maintenance of the old channels in the glasshouse areas Ik and Ig is lowered. The amount saved by this reduction equals the extra amount necessary to maintain the new canals. The lowering of the frequency is acceptable because these channels now only transport drain water from the sub-surface drainage systems. The cost for maintenance in these areas is rather high because it is partially carried out by hand, as there is no maintenance path. The cost of mechanical weed control is much lower for the new canals. An interesting aspect from the viewpoint of the Waterboard is that, if the maintenance of the existing channels in these areas is left out of consideration, a saving is obtained of about KDfl 10.0. The reason could be that the maintenance of these tertiary canals is to be carried out by the gardeners in the new situation. The investment cost of KDfl. 868.0 does not reckon with the fact that the removal of the polluted sediment will certainly be more expensive.

Recapitulating, it can be stated that the implementation of the proposed modifications will lead to a well functioning drainage system.

Variant 3, the present situation with a modified Canal A

This variant was chosen to be able to determine which of the modifications should be carried out first. It represent the existing situation as far as the glasshouse areas are concerned and a modified situation as far as the main drainage canals (nodes numbered 60 to 76) are concerned. This situation will occur in case it is decided to carry out the works at Canal A first, and in a later phase the modification plans in and around the glasshouse areas (Fig.5.10).

The score table 5.5 can be compared with the one of variant 1 but for some details because in this case the storage area as well as the total glass area in the glasshouse regions represent the present, and not the future situation. The score table which corresponds with the present situation and can be seen as the zero variant, does not differ very much from the score table of variant 1. It is not presented here for that reason. The differences between the two score tables are only noticeable in the areas Erica Ik and Ig. In the present situation, contrary to variant 1, both the steady and the non-steady 'severe' criteria are met.

Bearing this in mind and reviewing Table 5.5. it can be stated that the modification of Canal A has positive effects on the Canal A itself and results in plusses in the nodes numbered 60 to 73 and 75 instead of minuses for rainfall no.2. As such, it results in an improved drainage situation for the upstream situated agricultural areas and, less important, for the pumped industrial areas. The benefits in these areas are not made a point of further research within the framework of this thesis.

This variant does not result in any important improvement of the situation in the glasshouse areas. From this result it may be concluded that, from the viewpoint of a better drainage situation in the glasshouse areas, the investment of kDfl. 420. is not advisable if the drainage system in the glasshouse areas is not modified along with it.

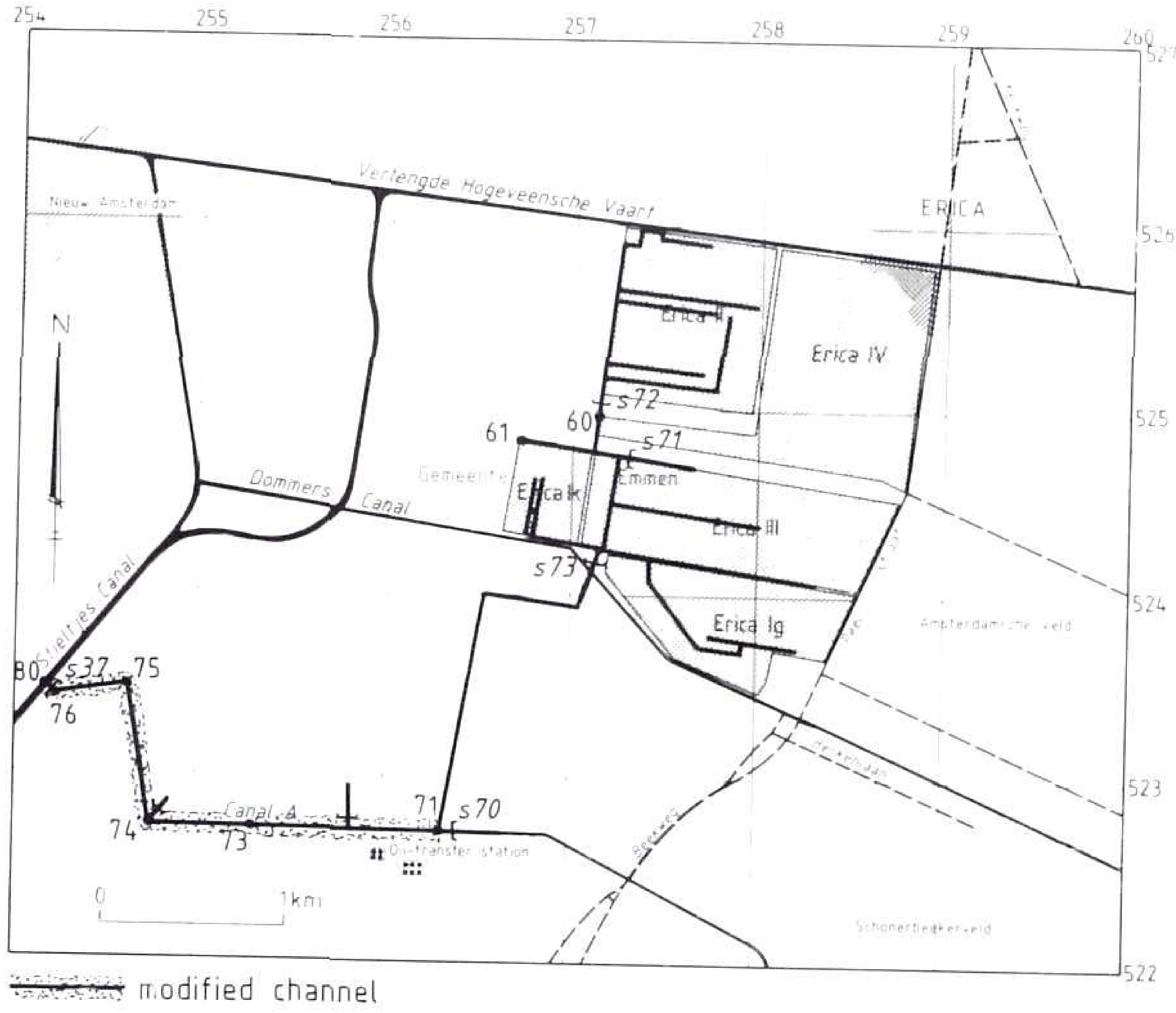


Fig.5.10. Schematization of the network of watercourses of the study area with the glasshouse areas in the present situation with Canal A modified according to variant 3.

Table 5.5. The evaluation of Variant 3, the present situation with a modified Canal A.

ode ber	Criteria rainfall no.1						Criteria rainfall no. 2						
	severe		more severe		severest		severe		more severe		severest		
2	+	+	+	+	***	***	+	+	-	-	***	***	Large agricultural areas
5	+	+	+	+	***	***	+	+	-	-	***	***	
7	+	+	+	+	***	***	+	+	-	-	***	***	
9	+	+	***	***	***	***	+	+	***	***	***	***	
11	+	+	+	+	-	+	+	+	+	+	+	+	Erica II
12	+	+	+	+	-	+	+	+	+	+	+	+	
13	+	+	+	+	+	+	+	+	+	+	+	+	
14	+	+	+	+	+	+	+	+	+	+	+	+	
15	+	+	+	+	+	+	+	+	+	+	+	+	
16	+	+	+	+	+	+	+	+	+	+	+	+	
17	+	+	+	+	+	+	+	+	+	+	+	+	
18	+	+	+	+	-	+	+	+	+	+	+	+	
19	+	+	+	+	-	+	+	+	+	+	+	+	
20	+	+	+	+	+	+	+	+	+	+	+	+	
21	+	+	+	+	-	+	+	+	+	+	+	+	
22	+	+	+	+	+	+	+	+	+	+	+	+	
23	+	+	+	+	+	+	+	+	+	+	+	+	
24	+	+	+	+	-	+	+	+	+	+	+	+	
25	+	+	+	+	-	+	+	+	+	+	+	+	
26	+	+	+	+	-	+	+	+	+	+	+	+	
27	+	+	+	+	+	+	+	+	+	+	+	+	
28	+	+	+	+	+	+	+	+	+	+	+	+	
29	+	+	+	+	+	+	+	+	+	+	+	+	
31	+	+	-	-	-	-	+	+	+	-	-	-	Erica Ik
32	+	+	-	-	-	-	+	+	+	-	-	-	
33	+	+	-	-	-	-	+	+	+	-	-	-	
34	+	+	-	-	-	-	+	+	+	-	-	-	
35	+	+	-	-	-	-	+	+	+	-	-	-	
36	+	+	+	-	-	-	+	+	+	-	-	-	
37	+	+	+	-	-	-	+	+	+	-	-	-	
40	-	+	-	-	-	-	+	+	+	-	-	-	Erica Ig
41	+	+	-	-	-	-	+	+	+	-	-	-	
42	+	+	-	-	-	-	+	+	-	-	-	-	
43	+	+	-	-	-	-	+	+	-	-	-	-	
44	+	+	+	+	-	-	+	+	+	-	-	-	
45	+	+	-	+	-	-	+	+	+	+	-	-	
46	+	+	-	-	-	-	+	-	-	-	-	-	
47	+	+	+	+	-	-	+	+	+	+	-	-	
48	+	+	-	-	-	-	+	+	-	-	-	-	
49	+	+	-	-	-	-	+	+	-	-	-	-	
50	+	+	+	-	-	-	+	+	+	-	-	-	
51	+	+	+	+	-	-	+	+	+	-	-	-	
52	+	+	+	+	-	-	+	+	+	+	-	-	
58	+	+	+	+	+	+	+	+	+	+	+	+	Erica III
59	+	+	+	+	+	+	+	+	+	+	+	-	
60	+	+	+	+	***	***	+	+	+	+	***	***	Central Drainage Canal
61	+	+	+	+	***	***	+	+	+	+	***	***	
62	+	+	+	+	***	***	+	+	+	+	***	***	
63	+	+	+	+	***	***	+	+	+	+	***	***	
64	+	+	+	+	***	***	+	+	+	+	***	***	
65	+	+	+	+	***	***	+	+	+	+	***	***	
66	+	+	+	+	***	***	+	+	+	+	***	***	External Drainage Canal
67	+	+	+	+	***	***	+	+	+	+	***	***	
68	+	+	+	+	***	***	+	+	+	+	***	***	
69	+	+	+	+	***	***	+	+	+	+	***	***	
71	+	+	+	+	***	***	+	+	+	+	***	***	Canal A
72	+	+	+	+	***	***	+	+	+	+	***	***	
73	+	+	+	+	***	***	+	+	+	+	***	***	
74	+	+	+	+	***	***	+	-	-	-	***	***	
75	+	+	+	+	***	***	+	+	+	+	***	***	
76	+	+	+	+	***	***	+	-	-	-	***	***	

Investment costs	[kDfl]	420.0
Maintenance costs	[kDfl]	60.0
Net present value of the costs	[kDfl]	5495.0

Variant 4

This variant can be compared with variant 3 as it concerns the present situation. The difference is, that in this case, the drainage situation in the glasshouse areas is modified in accordance with the plans of the Erica drainage working group, and not Canal A (Fig.5.11). The results, as presented in Table 5.6., demonstrate that the criteria in the glasshouse areas are met in that case. The situation becomes somewhat worse for Erica II for rainfall no.2 in comparison with variant 3, as in more places the 'severest' criteria are not met. Great improvement, however, is obtained for the Erica Ik and Ig areas. The situation with severe water clogging belongs to the past if the modification works in the glasshouse areas are carried out. Given that objective, it seems well-advised to invest the kDfl 448.0 in the first phase. The water levels in the main drainage canal from nodes numbered 60 to 76 however, still reach very high levels and the design criteria are not satisfied in all places . From that point of view it seems judicious to carry out the modification works on Canal A as well.

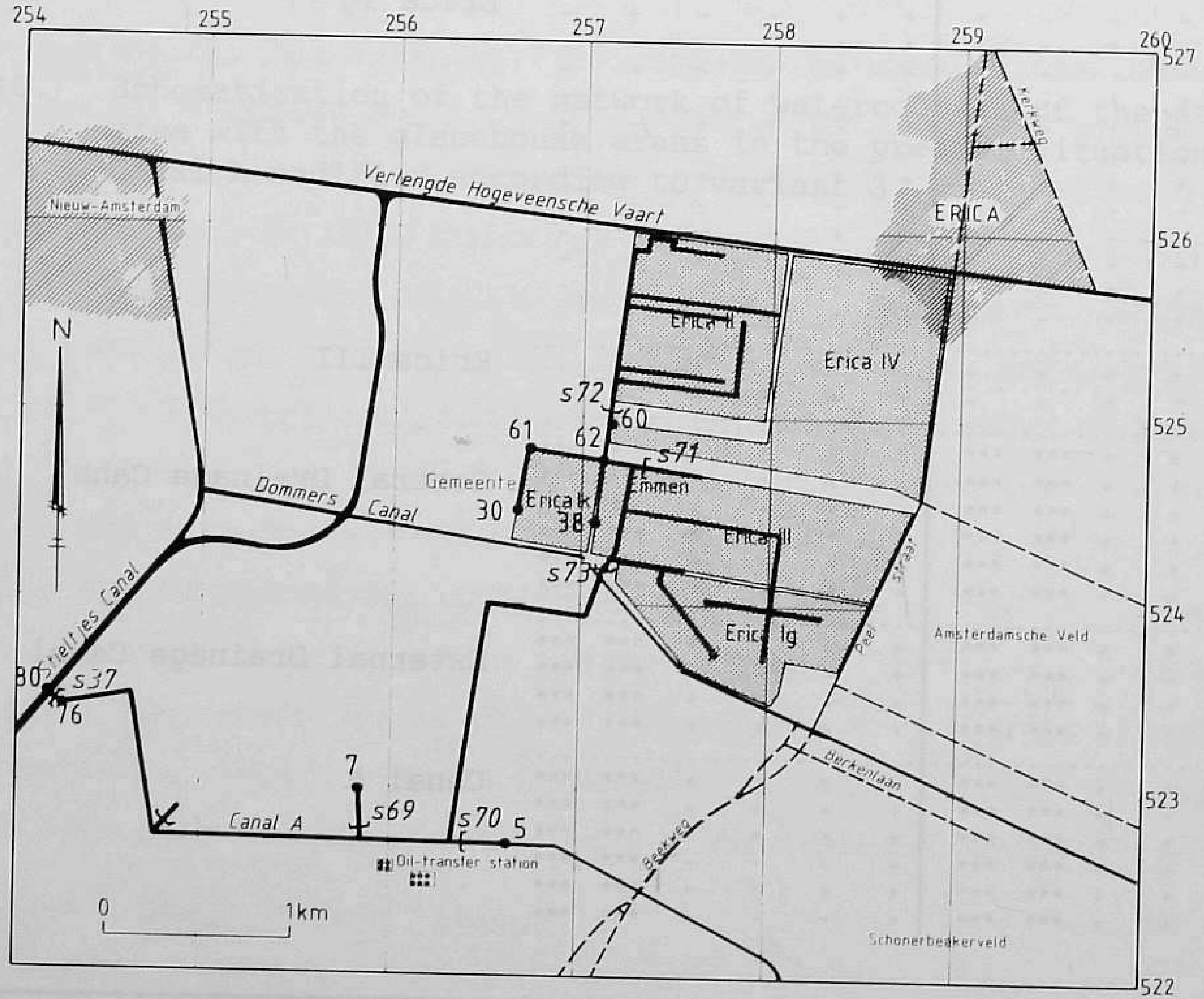


Fig.5.11. Schematization of the network of watercourses of the study area with the glasshouse areas modified according to variant 4.

Table 5.6. The evaluation of Variant 4, the drainage situation in the glasshouse areas is modified.

node number	Criteria rainfall no.1						Criteria rainfall no. 2						
	severe		more severe		severest		severe		more severe		severest		
[illegible]	+	+	+	+	***	***	-	-	-	-	***	***	Large agricultural areas
5	+	+	+	+	***	***	-	-	-	-	***	***	
7	+	+	+	+	***	***	+	+	-	-	***	***	
8	+	+	***	***	***	***	+	+	***	***	***	***	
11	+	+	+	+	-	+	+	+	+	+	+	-	Erica II
12	+	+	+	+	-	+	+	+	+	+	+	-	
13	+	+	+	+	+	+	+	+	+	+	+	-	
14	+	+	+	+	+	+	+	+	+	+	+	-	
15	+	+	+	+	+	+	+	+	+	+	+	-	
16	+	+	+	+	+	+	+	+	+	+	+	-	
17	+	+	+	+	+	+	+	+	+	+	+	-	
18	+	+	+	+	-	+	+	+	+	+	+	-	
19	+	+	+	+	-	+	+	+	+	+	+	-	
20	+	+	+	+	+	+	+	+	+	+	+	-	
21	+	+	+	+	-	+	+	+	+	+	+	-	
22	+	+	+	+	+	+	+	+	+	+	+	-	
23	+	+	+	+	+	+	+	+	+	+	+	-	
24	+	+	+	+	-	+	+	+	+	+	+	-	
25	+	+	+	+	-	+	+	+	+	+	+	-	
26	+	+	+	+	-	+	+	+	+	-	-	-	
27	+	+	+	+	+	+	+	+	+	+	+	-	
28	+	+	+	+	+	+	+	+	+	+	+	-	
29	+	+	+	+	+	+	+	+	+	+	+	-	
30	+	+	+	+	+	+	+	-	-	-	-	-	Erica Ik
38	+	+	+	+	-	+	+	-	-	-	-	-	
39	+	+	+	+	+	+	+	-	-	-	-	-	
49	+	+	+	+	-	+	+	+	+	+	-	-	Erica Ig
50	+	+	+	+	-	+	+	+	+	+	-	-	
51	+	+	+	+	+	+	+	+	+	+	-	-	
52	+	+	+	+	+	+	+	-	-	-	-	-	
48	+	+	+	+	-	+	+	+	+	+	-	-	
47	+	+	+	+	+	+	+	+	+	+	-	-	
53	+	+	+	+	-	+	+	+	+	+	-	-	
54	+	+	+	+	-	+	+	+	+	+	-	-	
55	+	+	+	+	-	+	+	+	+	+	-	-	
56	+	+	+	+	-	+	+	+	+	+	-	-	
57	+	+	+	+	+	+	+	+	+	+	-	-	
42	+	+	+	+	+	+	+	+	+	+	-	-	
58	+	+	+	+	+	+	+	+	+	-	-	-	Erica III
59	+	+	+	+	+	+	+	-	-	-	-	-	
60	+	+	+	+	***	***	+	+	-	-	***	***	Central Drainage Canal
61	+	+	+	+	***	***	+	+	+	-	***	***	
62	+	+	+	+	***	***	+	+	+	-	***	***	
63	+	+	+	+	***	***	+	-	-	-	***	***	
64	+	+	+	+	***	***	+	-	-	-	***	***	
65	+	+	+	+	***	***	+	-	-	-	***	***	
66	+	+	+	+	***	***	+	-	-	-	***	***	External Drainage Canal
67	+	+	+	+	***	***	-	-	-	-	***	***	
68	+	+	+	+	***	***	-	-	-	-	***	***	
69	+	+	+	+	***	***	-	-	-	-	***	***	
71	+	+	+	+	***	***	-	-	-	-	***	***	Canal A
72	+	+	+	+	***	***	+	+	-	-	***	***	
73	+	+	+	+	***	***	+	+	+	+	***	***	
74	+	+	+	+	***	***	+	-	-	-	***	***	
75	+	+	+	+	***	***	+	-	-	-	***	***	
76	+	+	+	+	***	***	+	+	+	-	***	***	

Investment costs	[kDfl]	448.0
Maintenance costs	[kDfl]	60.0
Net present value of the costs	[kDfl]	5703.0

Variant 5

With the result of variant 4 in mind, it is interesting to determine whether all modification works on Canal A should be carried out or whether, perhaps, the dredging works on the farthest part of that channel, containing polluted sediment, could be omitted or at least postponed. To answer that question the results of variant 5 are given in Table 5.7. This variant can be compared with variant 4, except for Channel A. The modifications as introduced in variant 2 are implied here as far as the concrete bridges are concerned and the dredging works up to node number 73 (fig.5.12) The results make clear that the situation is improved especially for Canal A as compared with variant 4. Because the water levels in the channel are lower the situation in the glasshouse areas too is somewhat improved.

From these results the conclusion may be drawn, that the investment of KDfl. 367.0 after the first investment of KDfl. 448.0 (resulting in a total investment of KDfl. 815.0) may be considered to be a sensible second step,

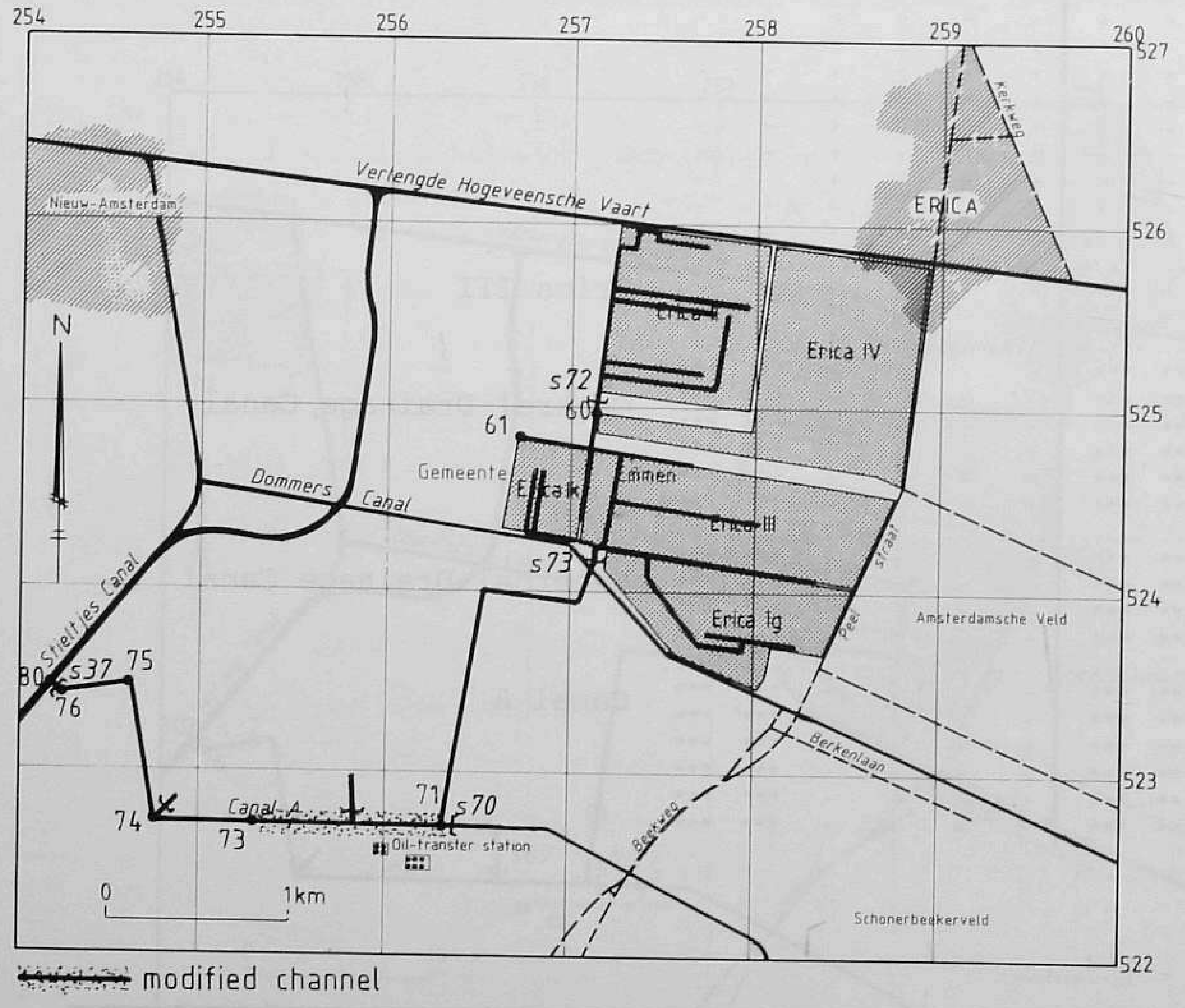


Fig.5.12. Schematization of the network of watercourses of the study area with the glasshouse areas and Canal A modified according to variant 5.

Table 5.7. The evaluation of Variant 5

node number	Criteria rainfall no.1						Criteria rainfall no. 2						
	severe		more severe		severest		severe		more severe		severest		
2	+	+	+	+	***	***	+	+	-	-	***	***	Large agricultural areas
5	+	+	+	+	***	***	+	+	-	-	***	***	
7	+	+	+	+	***	***	+	+	-	-	***	***	
8	+	+	***	***	***	***	+	+	***	***	***	***	
11	+	+	+	+	-	+	+	+	+	+	+	+	Erica II
12	+	+	+	+	-	+	+	+	+	+	+	+	
13	+	+	+	+	+	+	+	+	+	+	+	+	
14	+	+	+	+	+	+	+	+	+	+	+	+	
15	+	+	+	+	+	+	+	+	+	+	+	+	
16	+	+	+	+	+	+	+	+	+	+	+	+	
17	+	+	+	+	+	+	+	+	+	+	+	+	
18	+	+	+	+	-	+	+	+	+	+	+	-	
19	+	+	+	+	-	+	+	+	+	+	+	-	
20	+	+	+	+	+	+	+	+	+	+	+	+	
21	+	+	+	+	-	+	+	+	+	+	+	+	
22	+	+	+	+	+	+	+	+	+	+	+	+	
23	+	+	+	+	+	+	+	+	+	+	+	+	
24	+	+	+	+	-	+	+	+	+	+	+	-	
25	+	+	+	+	-	+	+	+	+	-	-	-	
26	+	+	+	+	-	+	+	+	+	-	-	-	
27	+	+	+	+	+	+	+	+	+	+	+	-	
28	+	+	+	+	+	+	+	+	+	+	+	+	
29	+	+	+	+	+	+	+	+	+	+	+	+	
30	+	+	+	+	+	+	+	+	+	+	+	-	Erica Ik
38	+	+	+	+	-	+	+	+	+	+	+	-	
39	+	+	+	+	+	+	+	+	+	+	+	-	
49	+	+	+	+	-	+	+	+	+	+	+	+	Erica Ig
50	+	+	+	+	-	+	+	+	+	+	+	+	
51	+	+	+	+	+	+	+	+	+	+	+	+	
52	+	+	+	+	+	+	+	+	+	+	+	+	
48	+	+	+	+	-	+	+	+	+	+	+	+	
47	+	+	+	+	+	+	+	+	+	+	+	+	
53	+	+	+	+	-	+	+	+	+	+	+	+	
54	+	+	+	+	-	+	+	+	+	+	+	+	
55	+	+	+	+	+	+	+	+	+	+	+	+	
56	+	+	+	+	+	+	+	+	+	+	+	+	
57	+	+	+	+	+	+	+	+	+	+	+	+	
42	+	+	+	+	+	+	+	+	+	+	+	+	
58	+	+	+	+	+	+	+	+	+	+	+	+	Erica III
59	+	+	+	+	+	+	+	+	+	+	+	-	
60	+	+	+	+	***	***	+	+	+	+	***	***	Central Drainage Canal
61	+	+	+	+	***	***	+	+	+	+	***	***	
62	+	+	+	+	***	***	+	+	+	+	***	***	
63	+	+	+	+	***	***	+	+	+	+	***	***	
64	+	+	+	+	***	***	+	+	+	+	***	***	
65	+	+	+	+	***	***	+	+	+	+	***	***	
66	+	+	+	+	***	***	+	+	+	+	***	***	External Drainage Canal
67	+	+	+	+	***	***	+	+	-	-	***	***	
68	+	+	+	+	***	***	+	+	+	+	***	***	
69	+	+	+	+	***	***	+	+	+	+	***	***	
71	+	+	+	+	***	***	+	+	+	+	***	***	Canal A
72	+	+	+	+	***	***	+	+	+	+	***	***	
73	+	+	+	+	***	***	+	+	+	+	***	***	
74	+	+	+	+	***	***	+	-	-	-	***	***	
75	+	+	+	+	***	***	+	+	+	+	***	***	
76	+	+	+	+	***	***	+	-	-	-	***	***	

Investment costs	[kDfl]	815.0
Maintenance costs	[kDfl]	60.0
Net present value of the costs	[kDfl]	5873.0

leading to a well functioning drainage situation as far as the hydraulic aspects are concerned.

5.4.4 Conclusions

The results of the simulations of the present and future situation make it clear that the drainage situation of the Erica glasshouse area is far from ideal. The situation will become even worse in future if no additional measures are taken with respect to the transport and storage of the runoff from the glass surfaces. Various variants have been evaluated, five of which were discussed in Section 5.4.3.

A situation is created where water clogging, flooding and serious damage belong to the past by providing additional storage and transport capacity in the glasshouse areas, combined with the modification of the profile of Canal A and the replacement of the existing culverts by concrete bridges in this channel. The advice to carry out the corresponding works takes the future situation into account as far as the glass surface area is concerned.

An important question for the realization of the works regards the timing of the various parts of the works. The polluted sediment on the bottom of a part of Canal A is a complicating factor. The simulations make it clear that it is advisable to begin with the modification of the drainage situation in the glasshouse areas. The resulting situation implicates drainage conditions that satisfy the design criteria for those areas. Even so some problems remain along Canal A. An advisable second step may be to modify Canal A except for the dredging works in the polluted part of the channel. That done, most problems are solved as far as the water levels are concerned. A situation is thus obtained that is comparable to the best solution, as presented by variant 2. This proposed phasing of the works allows for time to determine how to deal with the polluted sediments. It must be realized that such an intermediate situation should preferably not be prolonged, because it may be expected that the sediment will be transported downstream into the Stieltjes Canal. The third and last step will, by this reasoning, have to be the removal of the polluted sediment.

5.5. Discussion

In Chapter 4 the EWAS model was introduced, in the foregoing sections an application of EWAS in the Erica glasshouse area was described. This section is dedicated to reflections about the value and the performance of the EWAS model. Reflections about the value of the model refer to the qualities of the model, reflections about the performance refer to the application possibilities and the ease of use.

5.5.1 The value of the EWAS model

The EWAS model can be regarded as a combination of four sub-programmes. It combines the following four functions:

. the non-steady flow through a system of watercourses is described,with the possibility to include the most commonly used structures;
. the lateral inflow can be described to a satisfactory degree of flexibility and detail;
. the first approximation of the net present value of the costs can be determined in case of a new system and in case of an already existing system needing reconstruction;
. a flexible routine for evaluation purposes in which water levels are set against to design criteria and/or used for the purpose of evaluating the design criteria themselves.

In the following these four functions will be discussed.

As to the part of the programme that describes the non steady flow, discussions might concern the stability of the programme, the absence of more sophisticated routines for dividing the system of watercourses into nodes and branches, the routine chosen for the solution of the set of equations, etc. These aspects, however, will not be discussed because they merely concern the programme KNOTA which was adopted here and where these aspects were not changed because the performance of the programme gave no reason to do so. Stability, for example, is largely determined by the chosen time step in relation to the variation in branch lengths but also to the variation in storage capacity around each node and the variation of the lateral inflow. Stability can be increased by reducing the variation in branch lengths and storage capacities, thus increasing the number of equations to be solved at each time step. This obviously influences the computation time needed. The solution, though, does not account for the influence of the variation of the lateral inflow. Stability can also be increased by reduction of the chosen time step, with a similar effect on total computation time. This method does reduce the effects of the lateral inflow. In the applications of both the KNOTA programme and the EWAS programme a workable solution has always been found, choosing one of these two approaches in case of instability.

Reflections about the part describing the lateral inflow might concern the choice of the rainfall runoff model. It is stressed in Section 4.2.3 that with the chosen structure of the programme other models can be implemented easily. The selected approach where a set of two Nash cascades can be combined with a base flow has turned out to be simple, flexible and easy to work with. Other models like the non-linear reservoir model are expected to have the same qualities and may perform better. For this part of the programme it can also be said that the applications have not yet given reason to implement other models.

Viewing the economic evaluation it can be stated that the approach to the real costs is a very rough one. On the other hand, it was never meant to be a detailed project evaluation. Its purpose is to have a second way of distinguishing between variants, in addition to the hydraulic performance of the system under consideration. The simple comparison of the investment costs will not be sufficiently detailed in most cases. For that reason the economic evaluation procedure includes also the exploitation and maintenance cost. Interesting example of such a case has been described in Volp and Veraa (1986), striking the balance between investment costs and maintenance costs in the design of watercourses. The atrocle discusses the question whether it is possible to reduce the net present value of the costs for a given time period and interest rate, by designing for a lower level of maintenance and larger dimensions. A system of watercourses was designed with different levels of maintenance and was evaluated on economical grounds and hydraulic performance. Previously in this chapter a second example was treated were the EWAS model was used as a designing tool.

Discussions about the routine of comparing water levels with design criteria should concentrate on the choice of the set of two criteria

both concerning waterlevels. Velocities are not taken into consideration because they seldom give reason for re-dimensioning under normal Dutch circumstances. Here the choice is made to concentrate on the non-steady design criterion: waterlevels that may not be exceeded longer than a given period of time in combination with a maximum waterlevel. The reason is, that the more systems of water courses are designed using non-steady flow models, the more this type of criteria are needed. The simulations with the PRODU model make clear that taking into account non-steady design criteria may lead to more refined design.

5.5.2 The performance of the EWAS model

The notion 'performance of the EWAS model' refers to its application possibilities and ease of use. The application of the model to the case of the Erica glasshouse area will be used to judge the performance. Two important stages can be distinguished in the application: the calibration and validation stage and the stage of the development and evaluation of variants. In the following the qualities of the model will be discussed in these two stages.

The calibration and validation stage.

In the calibration and validation stage the addition of the rainfall runoff module has proven to be a valuable extension to the non steady flow programme. Usually the lateral inflow to the system is determined a priori, based on knowledge of the behaviour of similar areas. The inflow is given as a load to the system in m^3/s. A change in this input must be accomplished by the substitution of this inflow on all nodes of the system concerned. In this way, however, the calibration process, directed at harmonization of the calculated and measured values of water levels and/or discharges becomes quite laborious. Flexibility and ease of use are important aspects in this stage and have a positive influence on the quality of the calibration. The possibilities to vary the rainfall runoff characteristics in the model have enhanced the understanding of the problematic nature of the drainage process in the area. The same holds for the possibility to study the effects of other rainfall quantities and of variations in the intensities during the rainfall event. Both aspects play an important role in the initial stage of the calibration process. Thus, the importance of the influence of the rainfall runoff relation in comparison to the influence of other variables is made clear. This does not only apply to the runoff from the glasshouses, as has been illustrated in the case described. The high degree of flexibility in determining the effects of various rainfall runoff characteristics has accelerated the calibration procedure. Summarizing it can be stated that the addition of the rainfall runoff module has a positive influence on the quality of the calibration as well as on the duration of the calibration.

It has been shown that, in order to carry out a correct simulation of water levels and discharges much attention must be paid to the detailed description of in- and outflowing quantities of water, such as the rainfall-runoff relationships in the area under consideration, but also to the storage possibilities in the system. The temporary storage in the ground dependent on the water level in the channel, is important for the correct simulation of water levels in the system. The approach followed here to simulate the temporary storage is rather simple but gave satisfactory results. A more refined approach to this temporary storage as well as to the rainfall runoff relationships can be found through linkage with the PRODU model in every node. This would require much more data and the resulting model would not be very distinct due to the greater number of parameters. Moreover, the model will be much slower and will require much more calculation time. In the following chapter the decision of not linking PRODU and EWAS will be discussed.

The evaluation procedure with the score table plays a role of minor importance in this stage. The same holds true for the economic evaluation procedure.

The stage of the development and evaluation of variants.

It is in this stage that the addition of all of the three modules has proven to be of value, especially with respect to the easiness of use. Though this can not be quantified it is likely to presume that, because of the userfriendliness, the addition of these three modules have a positive influence on the quality of the variants developed.

The rainfall runoff module offers the possibility to study the effects of different design storms. The ease of use is even an incentive to do so, due to the influence on the quality of the results. As a second advantage the possibility must be mentioned to change the land use by changing the rainfall runoff characteristics of the area. By this method the influence of changing land use in future can be incorporated in the results.

The score table evaluation module has shown to be of particular value in the design process. This presentation of the degree to which the design criteria are met stimulates investigation into the matter of whether alternative solutions may obtain better results or the same results at lower costs. It is assumed that, applied in this way, the addition of this module has a positive influence on the quality of the variants developed, as well as on the duration of the variant development stage. The fact that in its application the design criteria can be related to land use or to return periods is an example of the flexibility of this form of evaluation. Although the use of the module where the design criteria are evaluated is not demonstrated, it may be expected that this offers more possibilities to obtain optimal results.

The significance of the economic evaluation module has been mentioned above. Especially the combination with the score table turns it into a valuable instrument for the development of variants.

CHAPTER 6 *DISCUSSION*

6.1 Introduction

In the previous chapters the background and application possibilities of two computer programmes have been discussed. Both the strong and weak aspects of the programmes were carefully thought about. This final chapter is the place to focus attention to the relevance of the work for practical purposes. In order to place the subject of this thesis in a wider context a view will be given on expected future developments.

6.2 ***The relevance of the work for practical purposes***

The first point of attention in the introduction of this chapter relates to the relevance of the work for the practical design purposes. It is very difficult to give an unequivocal answer to this question. It is possible, however, to summarize a number of points based on experiences with *EWAS* and *PRODU* which lead to recommendations for the design practice.

The aim of the study as described in Chapter 1, was the development of a tool for the designer, to be used for the generation and evaluation of design variants of a water management system with emphasis on the hydraulic qualities and the economic consequences of the design. As result of this choice two models have been developed. The PRODU model was developed to determine the relationship between a particular form of water management and the agricultural production. The EWAS model is especially suitable as an instrument for the designer of the infrastructure of watercourses. It can be applied as a tool for testing the new system with a view to its hydraulic qualities. The link with the economic module results in an optimal design, where full use can be made of the dynamic storage in the system. As such, the model is especially suitable for networks of water courses in which non-steady phenomena are important.

Experiences with EWAS have shown that the combination of a score table with an economic evaluation procedure, results in an uncomplicated and lucid assessment method for the design process. In addition it has been shown that the inclusion of the rainfall-runoff relationship in a non-steady calculation routine together with the +/- table, provides a good combination for exploring the different sensitivities. The resulting recommendation is therefore to include these elements in such kinds of models. The +/- table in EWAS is only used for testing the water levels, but its use need not be limited to this. Depending on the situation other variables can be considered, such as flow velocity and water quality parameters.

Experiences with EWAS have again stressed the importance for the design of knowledge concerning rainfall-runoff behaviour for a certain area. The rainfall-runoff behaviour is highly dependent on the situation. The resulting recommendation from this relates not so much to the design practice, but rather to water management practice. Because current knowledge about the rainfall-runoff process in a particular case is often not available, it is recommended to include the following objective in the measuring plan as part of the water quantity management plan: to carry out observations to determine the rainfall-runoff relation of the most important sub-areas in the region.

When applying non-steady calculation routines in the design of networks of watercourses in which the dynamic storage of the system is used, non-steady design criteria become essential. Experiences in Erica have shown that in that case there is a need for guidelines not only with regard to the level exceedances of short duration, but also for those of much longer duration. This applies not only to criteria for water levels, but also to other variables such as the above mentioned water quality parameters.

The statistical procedure for analyzing the results of simulations with PRODU as proposed in Section 2.4 has proven to be of value for evaluating various water management scenarios.

With the applications of PRODU it has been demonstrated that there are possibilities for obtaining the formulation of non-steady criteria with PRODU-type models. For design practice this means that there is still much to be done to formulate non-steady criteria for a number of soil types and crops.

The results of PRODU have emphasized that in optimizing water level management, all aspects should be carefully included in the consideration. In the choice of a date to switch from winter to summer level for example, it has been made clear that an optimization with respect to workability conditions in spring should be combined with the consequences for the moisture availability during the growing season. The example is based on a situation without sprinkling possibilities, emphasizing that optimal conditions for soil cultivation activities in spring should ideally only be aimed at in a situation where the water supply later later that summer is optimally assured as well, for example through the use of sprinkling devices. If the weight of the two kinds of damage, the 'wet' and the 'drought' damage, must be reassessed, the example of the relation of the open water percentage and potato yield has emphasized the importance of an optimization with respect to workability conditions.

6.3 Design and operation of water management systems: a view to the future.

The field of design and operation of water management systems covers a wide range of subjects. It is impossible to treat more than a few subjects to some degree of detail. Therefore the title of this thesis indicates that only a limited number of aspects are treated. The choice for the subjects considered and the approach followed is based on a personal affinity to the design practice on the one hand, and on the backgrounds of the research group of the university on the other hand.

In this section some space is devoted to developments in water management, mainly with a view to pointing out where to place this thesis in a wider context. This view is based on recent developments in the Netherlands, and it indicates the complexity of the field of interest. This does not mean that the value of the observations is restricted to the Netherlands. It can be stated that the thread in the observations holds for more countries in the world, though the point in time as well as the time scale may differ.

Broadly speaking it can be stated that until recently the attention of research and practice with respect to water management was focussed merely on the quantities of water that had to be transported. In the Netherlands attention was predominantly paid to the discharge of the excess water, in other countries water supply was generally more important. The dimensioning of the watercourses and the necessary structures can be seen as the common characteristic. Attention for water quality aspects was restricted and, certainly in the Netherlands, predominantly aimed at the problems related to the salt content.

In the period after 1970 various developments started, some on a national and regional level, others on a local level. For this overview of the developments this differentiation between two levels is not made because it becomes less predominant with time and will further diminish in the future.

The perception arose that by striving at posthaste discharge of water, the chance of drought damage is introduced or increased. This stimulated the need for research into the relation between water management and drought damage: the first step towards the developments of later years. In this stage the research activities of the first PAWN study took place. With the development of instruments for optimal allocation of water, the quality of the water gradually drew more attention. The first optimization of the allocation of water focussed on the place and time, later the procedure was further refined by adding the differentiation of the quality of the water that could be allocated. The same holds for the differentiation between the various water users. Nature, forest and landscape became important users of water for whom the water quality aspects were of particular interest. More recently the sediments on the bottoms of the water systems drew attention, not only with respect to the quality of the sediments itself but also regarding the relation to the processes in the water. It became more and more clear that the various systems related to the water should be considered in combination the mutual relations.

The field of water management became more complex as well as more comprehensive. For research and practice is the integrated approach that is to be followed in future has become important. This integrated approach makes it necessary for the methods and instruments applied or to be developed to become more complex. To an increasing degree this complexity complicates the formulation of the policy to be followed in future. Even the description of the actual condition of the various systems related to water management will require a major effort. For the formulation of the policy in water management it then becomes important to determine what effects may be expected from certain policy measures: the so-called 'scenario development'. Furthermore, the monitoring of the actual condition of the system will require an important effort. The importance of a monitoring programme can be derived from the determination of the efficiency of policy measures taken in the past.

In future the attention of research and practice in the field of water management must especially be focussed on the following fields:

1. Much attention must be given to research into the interrelations of the various systems. This does not only apply to the determination of the relations of water and water management and the 'yields' of the various water users, but also of those between the water users themselves and the intensity of their mutual relations. This formulation is open ended in order to prevent exclusion of certain aspects. The developments in agriculture as well as ecology for example can be expected to have an important influence on the water management policy in future.
2. Attention must be paid to the determination of the characteristics of the systems under consideration. In consequence of the growing complexity and detail of the descriptions of the systems and their mutual relations, the need for more detailed basic data of high quality becomes more urgent. It is to be expected that this will require much effort especially because part of these data will have to be determined using laboratory analysis, or through inventarization in the field. It is important to note here that, limited to the determination of these basic data, it is indispensable to give attention to the transferability and the accessibility of the data, in order to assure optimal use.
3. The monitoring of the condition of the system will become increasingly important. As far as the transferability and the accessibility are concerned the same holds for the data obtained by this activity. In addition it may be expected that new monitoring techniques will become operational. Particularly remote sensing techniques will probably play an important role, especially regarding the cost-aspects of these

monitoring techniques.

4. The development of Decision Support Systems (DSS) and Expert Systems (ES) in the field of water management should be mentioned. In consequence of the increasing number of parties concerned in well considered water management, the development of DSS and ES will be of great value to operational water management in the weighting of the effects of various measures.
5. It must be noted that the development of presentation techniques for research results as well as part of the above mentioned DSS and ES will become increasingly important. Because research will cover more aspects of variable character results of such studies will be more difficult to interpret. However the need for translation of the results into future water management policy makes it essential that the results be presented correctly and completely. Much attention must be paid to the presentation of the relations between the uncertainty in model parameters and the effects of these uncertainties on model results, and these effects will have to be translated into margins for the policies to be formulated. The same holds for the effects of the presented measures and the scenarios which have been investigated.
6. The same development of increasing integral approach will be found in the design practice of water management systems. In the design a larger role will be played by the relations between the various interested parties and by the relations between water quality and quantity aspects. This will be expressed in the design criteria but also in a greater influence of future management strategies on the design. The application of new presentation techniques and the use of digitized data will further increase. Computer aided design and drafting techniques which are common in other technical fields will be more and more applied in the design of water management systems.

There will surely be fields of interest for research and practice in water management which have not yet been mentioned, though the most important have been covered. There is one exception to be mentioned here, which may turn out to be the most important item. What is meant is the establishment of an administrative, organizational, financial, juridical infrastructure which may differ from the existing and better equipped to shape a valuable integral water management. This means that a situation must be created enabling an optimally equipped administrative body to really give shape to

the conception of integral water management. Without going into details it seems not unthinkable that the present situation in the Netherlands, involving three ministries, provincial governments, water boards and local governments, is eligible for improvement. The form in which that should be realized is still unclear but obviously something must be done in the near future. The deterioration of the environment has elucidated that only a world wide approach will result in optimal, if not barely acceptable effects. The same may hold for water management. This may not necessarily have to be on a global scale but with the establishment of the first step to the United Nations of Europe in 1992 a more than nation wide approach is not unlikely and in any case highly desirable.

Consideration of the above makes it clear that this thesis contributes only to a limited extent to what is going on in the field of water management and merely touches on what may be expected in future. Partly due to the developments which took place after the start of the study this thesis contributes predominantly to the developments mentioned under item six.

SUMMARY AND CONCLUSIONS

This thesis deals with a number of aspects important for designing water management systems. The design of a water management system can be defined as an illustrated or written plan showing the dimensions of water courses, pumps, dams and other constructions essential to the efficient management of water in a particular area. The design takes *into account all interests of the parties involved in the area. It is* essential to emphasize that the term "design" refers to a plan which is generally presented to others for further consideration. The title of *the thesis also indicates that only a limited number of aspects* important for the design are considered, certainly not all the aspects.

In the course of this study, two main areas of attention can be distinguished: firstly an area where the emphasis lies on water level management and secondly, one where the accent lies on the dimensioning aspect.

Water level management

In the first main area under consideration the emphasis lies on water management aspects. Here the accent is on the consequences of certain water management measures for agricultural production, and especially *for potatoes, here used as the standard crop in cultivation. The term* "management measures" refers to all the measures involved in the choice of water level, the acceptability of level exceedances and the level variation over the whole year. The effect of a particular policy towards water management depends amongst other things on the time of year, the crop and the extent of level exceedances. In order to estimate the consequences of a certain form of water management practice, a model has been developed and is described in Chapter 2. The model is named PRODU which stands for ' a model for the determination of the **PRODU**ction of agricultural areas in relationship with water management'. To illustrate its application possibilities two examples are described in Chapter 3.

The model describes the development of the crop during the year and the potential and actual production in relation to the water level management during that year. The basis for this is the water movement through the saturated and unsaturated zone. The procedure which determines the level in the surface water takes into account all aspect of the water management policy regarding the planned levels and the supply and drainage possibilities.

Important factors for describing crop development are the dates of planting and emergence. The date of planting is determined after the land has been cultivated in spring. This date depends on the workability constraints which depends on the moisture content of the top layer of soil. The emergence date is determined on the basis of the necessary heat sum based on the temperature near the germinating seed. The moisture content of the soil is also important in this case. Once the crop has emerged, its development both above and below the ground is linked to the development stage. This development stage is determined by the germination, harvest and actual dates.

Potential and actual production is calculated on the basis of the potential and actual transpiration. Potential transpiration is determined by the development of the plant and meteorological conditions. The actual transpiration and production depend on the extent to which the plant has access to moisture from the soil. Ensuring the latter by means of the unsaturated and saturated zones is a task of water level management.

By analysing the results of simulations using this model carried out over a year, various forms of management can be compared. A procedure for the analysis is proposed whereby in a certain year actual production with one form of management is compared to the highest potential yield for that same year of all the management forms of the comparison.

In Chapter 3 two applications of the model are described by way of example. The following can be concluded:

* Because the statistical analysis of the results as proposed in paragraph 2.4 gives a good insight into the differences, this is a satisfactory means of evaluating various water management scenarios.
* In the process of optimising water level management it is important that all aspects are carefully considered. In the choice of a date to switch from a winter to a summer level for example, it has been made clear that the optimization should not be pointed at the workability conditions in spring only, but should preferably include the moisture availability during the growing season as well. The example in paragraph 3.2 is based on a situation with no sprinkling possibilities emphasizing all the more that optimal conditions for workability in the spring can only be aimed at if the water supply during the growing season is assured.
* If the importance of the two kinds of damage must be re-assessed, the example of the relation of the open water percentage and potato yield has emphasized the importance of an optimization with respect to workability conditions.
* With PRODU-type models non-steady design criteria can be formulated. Before these can be applied, however, a number of simulations for different crops and soil types will have to be carried out.

Dimensioning

The evaluation of the different variants for the design is a central aspect in the second area of attention, in so far as this concerns the dimensioning of water courses, pumps and other provisions. The most important aim was to develop a procedure which makes it possible to develop and evaluate variants for a water management system in a simple and intelligent way, taking into account the various interests in the management area in question. The procedure described in Chapter 4, in the form of the EWAS model includes the evaluation of the hydraulic characteristics of the proposed plan and a presentation of the financial consequences if this plan is carried out. EWAS is an acronym of 'a model for the design and **E**valuation of **WA**ter management **S**ystems'. The evaluation of the hydraulic characteristics involves the testing of the calculated water levels on the design criteria set for this design. The procedure described is flexible in a number of ways: it is possible to apply non-steady criteria to the design as well as steady criteria. Provision has also been made for the case where various parties interested in one section of the whole area have different requirements for water levels occurring under dimensioning circumstances. A further possibility is an application whereby it is not a question of various interested parties presenting different requirements, but where a number of design criteria are used with different times of return. The question whether the norms are satisfied or not is then related to an acceptation limit linked to the frequency of occurrence.

The economic evaluation procedure determines the net present value of the cost of each part of the project on the basis of uniform prices, per metre or per cubic metre. Besides the investment costs, it also takes the maintenance and exploitation costs into account. If a nominal yield from the sub-areas can be supplied, this procedure will determine the internal rate of return in addition to the total net present value of the cost of the whole project.

The evaluation of the hydraulic characteristics is presented in a score table in combination with the net present value of the cost of the proposed system. This form of presentation makes it easy to assess the extent to which the design satisfies the applied criteria and to relate this to the costs of the system. This assessment can lead to changes in the design resulting in a new variant for the final design or in an improvement of the variant concerned.

It may be that the assessment results in an adaptation of the applied criteria. It is possible, for example, that an interested party, when presented with the financial consequences, may adapt his requirements for the design. In this way the norms are evaluated.

In Chapter 5 an application of the EWAS model is described. The following observations can be made in connection with it:

* The procedure described above works very well as a method of assessment in the design process.
* The including of rainfall-runoff relations in a model for non-steady fluid transport through a system of water courses in combination with the score table provides a good method of exploring the various sensitivities. The resulting recommendation is therefore to include both these modules in these sorts of models.
* The score table is only used in this thesis to test water levels, but need not be limited to this. Depending on the situation other variables may be considered, such as flow rates and water quality parameters.
* Application in the glasshouse area Erica has emphasised the importance of the knowledge of rainfall-runoff behaviour of the area concerned when considering the design. It often appears that such knowledge is not available in a particular case. For this reason, as one of the objectives in the measurement plan which is a part of the overall water management plan, the following recommendation is made: measurements on hydrologic parameters should be carried out in order to determine the rainfall-runoff relations of the most important areas within the management area.
* By applying non-steady calculation methods in the design of water course networks in which the dynamic capacity of the system is used, the need will arise for non-steady criteria. The application to the Erica glasshouse area have taught that in such a case guidelines are required with regard to both short-term and for longer-term water level exceedances. This does not exclusively concern water level norms, but may also other apply to other variables such as water quality parameters.

SAMENVATTING EN CONCLUSIES

Dit proefschrift behandelt een aantal aspecten die van belang zijn bij het ontwerp van waterbeheerssystemen. Het ontwerp van een waterbeheerssysteem kan gedefinieerd worden als een getekend en/of beschreven plan met dimensies van waterlopen, pompen, stuwen en andere noodzakelijke voorzieningen om op efficiënte wijze het water in een gebied te kunnen beheren. Het ontwerp houdt rekening met de belangen van alle betrokkenen in het gebied. Essentieel bij het begrip 'ontwerp' is, dat het een plan betreft, dat over het algemeen aan anderen gepresenteerd wordt ter verdere overweging. De titel van het proefschrift geeft verder aan dat slechts een aantal aspecten aan de orde komen die van belang zijn bij het ontwerp, zeker niet alle aspecten.

Bij de behandeling van dit onderwerp in dit proefschrift kunnen twee hoofdaandachtsvelden worden onderscheiden: één waarbij de nadruk ligt op het peilbeheer en één waarbij het accent ligt op het dimensioneringsaspect.

Peilbeheer

Bij het eerste hoofdaandachtsgebied ligt de nadruk op de beheersaspecten. Hierbij ligt het accent op de gevolgen van bepaalde beheersmaatregelen voor de landbouwproductie, met name voor het in dit geval gehanteerde standaard akkerbouwgewas aardappelen. Onder beheersmaatregelen wordt hier het geheel verstaan van peilkeuze, de toelaatbaarheid van peilsoverschrijdingen maar ook het peilsverloop gedurende het jaar. Het effect van een bepaalde keuze van beheer is onder meer afhankelijk van *de tijd van het jaar, het gewas en de mate van peilsoverschrijding.* Om de gevolgen van een bepaalde beheerspraktijk te kunnen inschatten is een model ontwikkeld, PRODU genaamd, dat is beschreven in hoofdstuk 2. PRODU is een acronym voor ' a model for the determination of the **PRODU**ction of *agricultural areas in relationship with water management.* Bij wijze van voorbeeld zijn in hoofdstuk 3 een tweetal toepassingen van PRODU beschreven.

Het model PRODU beschrijft de ontwikkeling van het gewas gedurende het jaar en bepaald onder meer de potentiële en actuele productie op basis van de berekende potentiële en actuele transpiratie gedurende dat jaar. Bij de berekening van de transpiratie wordt uitgegaan van de waterbeweging door de onverzadigde en onverzadigde zone. Door daarbij het open water peil te betrekken ontstaat de relatie tussen gewasopbrengst en peilbeheer. Voor de bepaling van het peil in het oppervlakte water wordt uitgegaan van de wijze van beheer met betrekking tot de na te streven peilen en de aan- en afvoercapaciteiten.

Belangrijk voor de beschrijving van de gewasontwikkeling zijn de plantdatum en de opkomstdatum van het gewas. De plantdatum wordt bepaald nadat de grond in het voorjaar is bewerkt. De bewerkingsmogelijkheden hangen af van een berijdbaarheidscriterium dat is gekoppeld aan het vochtgehalte in de bovenste laag van het profiel. De opkomstdatum wordt bepaald op basis van een warmtesom uitgaande van de temperatuur in de nabijheid van het kiemende zaad. Ook hierbij is het vochtgehalte in de bovenste laag van de grond van belang. Nadat het gewas is opgekomen wordt de ontwikkeling van het gewas zowel boven als ondergronds, gekoppeld aan het ontwikkelingsstadium. Dit ontwikkelingsstadium wordt bepaald door de opkomstdatum, de oogstdatum en de actuele datum.

De potentiële en actuele productie wordt berekend op basis van de potentiële en actuele transpiratie. De potentiële transpiratie wordt bepaald door de ontwikkeling van de plant en de meteorologische omstandigheden. De actuele transpiratie en productie wordt sterk bepaald door de mate waarin de plant kan beschikken over het bodemvocht. Dit laatste is via de onverzadigde en verzadigde zone een functie van het peilbeheer.

Door de resultaten van langjarige simulaties met dit model te analyseren kunnen verschillende vormen van beheer met elkaar worden vergeleken. Voor de analyse wordt een procedure voorgesteld waarbij de actuele productie in een bepaald jaar voor een bepaalde vorm van beheer wordt vergeleken met de hoogste potentiële opbrengst voor dat jaar van alle in de vergelijking betrokken vormen van beheer.

In hoofdstuk 3 zijn bij wijze van voorbeeld een tweetal toepassingen van het model beschreven. Naar aanleiding daarvan kan het volgende worden geconcludeerd:

- De statistische analyse van de resultaten van simulaties met PRODU, zoals voorgesteld in paragraaf 2.4 geeft een goed inzicht in de effecten van de verschillende vormen van peilbeheer op de landbouwopbrengsten. De procedure deze zich goed voor het afwegen van verschillende beheerssenario's.
- Bij de optimalisering van het het peilbeheer is het van belang om zorgvuldig alle aspecten in de beschouwing op te nemen. Bij de keuze voor de overgangsdatum van winter naar zomerpeil bijvoorbeeld, verdient het aanbeveling om niet alleen te optimaliseren naar bewerkingsomstandigheden in het voorjaar, maar om ook de vochtvoorraad gedurende het groeiseisoen daarbij te betrekken. In het voorbeeld dat is beschreven in paragraaf 3.2 is uitgegaan van een situatie zonder beregening, waardoor des te sterker werd benadrukt dat optimale omstandigheden voor de grondbewerking in het voorjaar bij voorkeur alleen nagestreefd moeten worden als de toediening van water ook optimaal kan worden gerealiseerd, bijvoorbeeld door beregening.
- In de afweging welke van de twee vormen van schade in de afweging het belangrijkste zal in sterke mate afhankelijk zijn van de bodemeigenschappen. In het voorbeeld beschreven in paragraaf 3.3 blijkt dat de bewerkingsomstandigheden belangrijker zijn dan de droogteschade gedurende het groeiseizoen.
- Met PRODU kunnen niet-stationaire normen worden geformuleerd. Voordat deze kunnen worden toegepast zullen vrij veel simulaties voor verschillende gewassen en grondsoorten moeten worden uitgevoerd.

Dimensionering

Bij het tweede hoofdaandachtsgebied staat de evaluatie van de verschillende varianten voor het ontwerp centraal, voor zover het de dimensionering van de waterlopen, pompen en andere voorzieningen betreft. Het belangrijkste doel daarbij is geweest een procedure te ontwikkelen die het mogelijk maakt om op eenvoudige en inzichtelijke wijze verschillende varianten voor een ontwerp van een waterbeheerssysteem te ontwikkelen en te evalueren, rekening houdend met de verschillende belangen in het beheersgebied. De procedure in de vorm van model EWAS, een acronym van 'a model for the design and **E**valuation of **WA**ter management **S**ystems', die wordt beschreven in hoofdstuk 4, omvat de evaluatie van de hydraulische eigenschappen van het voorgestelde plan en een presentatie van de financiële consequenties als dit plan tot uitvoering wordt gebracht. In hoofstuk 5 is ter illustratie een toepassing van EWAS beschreven.

De evaluatie van de hydraulische eigenschappen bestaat uit de toetsing van de berekende waterstanden aan dimensioneringsnormen die voor een bepaald ontwerp zijn opgesteld. De beschreven procedure is op verschillende manieren flexibel gehouden in haar opzet: het is mogelijk naast stationaire normen niet-stationaire normen op te leggen aan het ontwerp. Daarnaast is voorzien dat meerdere belanghebbenden voor eenzelfde onderdeel van het gehele gebied verschillende eisen stellen ten aanzien van de waterstanden die mogen optreden onder dimensioneringsomstandigheden. Ook is een toepassing mogelijk waarbij niet verschillende belanghebbenden verschillende eisen stellen, maar waarbij meerdere dimensioneringsnormen zijn gebruikt met verschillende herhalingstijden, waarbij het wel of niet voldoen aan deze normen is gerelateerd aan een acceptatiegrens die verband houdt met de frequentie van voorkomen.

Het economisch onderdeel van de procedure bepaalt op basis van eenheidsprijzen, per strekkende meter of per kubieke meter, de netto contante waarde van de kosten voor elk van de onderdelen van het project. Hierbij worden naast de investeringskosten, de onderhouds- en exploitatiekosten in de beschouwing betrokken. Indien een nominale opbrengst van de inliggende gebieden kan worden gegeven bepaalt deze procedure naast de totale netto contante waarde voor het gehele project ook de interne rentevoet.

De evaluatie van de hydraulische eigenschappen wordt gepresenteerd in een overzichtelijke score tabel in combinatie met de netto contante waarde van de kosten van het voorgestelde systeem. Deze presentatievorm maakt een snelle beoordeling mogelijk, waarbij de mate waarin het ontwerp voldoet aan de opgelegde normen gemakkelijk kan worden gerelateerd aan de kosten van het systeem. Deze beoordeling kan aanleiding geven tot veranderingen van het ontwerp resulterend in een nieuwe variant voor het uiteindelijke ontwerp of in een verbetering van de betreffende variant.

Daarnaast is het mogelijk dat de beoordeling aanleiding geeft tot het bijstellen van de opgelegde dimensioneringsnormen. Het is zeker niet onvoorstelbaar dat een belanghebbende, geconfronteerd met de financiële consequenties van zijn eisen voor het ontwerp in een bepaald geval bereid is deze aan te passen. Op deze manier worden de normen geëvalueerd.

In hoofstuk 5 is ter illustratie een toepassing van de procedure beschreven. Naar aanleiding daarvan kan het volgende worden opgemerkt:

* De hierboven beschreven procedure is een zeer plezierig werkende beoordelingsmethode bij het ontwerpproces;
* Het opnemen van de neerslag-afvoer relaties in een niet-stationair rekenmodel samen met de scoretabel resulteert in een goede combinatie voor het aftasten van de verschillende gevoeligheden. De aanbeveling die hieruit volgt is dan ook om deze beide modules op te nemen in dit soort modellen.
* De scoretabel is in dit proefschrift alleen gebruikt voor het toetsen van waterstanden, maar daartoe hoeft het zeker niet beperkt te blijven. Afhankelijk van de situatie komen ook andere variabelen in aanmerking, zoals stroomsnelheden en kwaliteitsparameters.
* De toepassing op het glastuinbouwgebied Erica heeft eens te meer aangetoond, hoe belangrijk de kennis omtrent het neerslag-afvoergedrag van een bepaald gebied is voor het ontwerp. Vaak blijkt dat deze kennis in een concreet geval ontbreekt. De aanbeveling die hieruit voortvloeit voor de beheerspraktijk is om als één van de doelstellingen bij het meetplan, behorende bij het waterkwantiteitsbeheersplan, op te nemen: het doen van waarnemingen ten behoeve van de bepaling van de neerslag-afvoer relatie van de belangrijkste gebieden binnen het beheersgebied.
* Door de toepassing van niet-stationaire rekenmethoden bij het ontwerp van stelsels waterlopen waarbij gebruik gemaakt wordt van de dynamische berging in het systeem, zal een behoefte aan niet-stationaire normen ontstaan. De ervaringen in Erica hebben geleerd dat er in dat geval behoefte is aan richtlijnen ten aanzien van de kort durende peiloverschrijdingen van enkele uren, maar zeker ook voor de veel langer durende peiloverschrijdingen. Overigens geldt dit niet uitsluitend voor normen ten aanzien van de waterstanden, maar zeker ook voor andere variabelen zoals waterkwaliteitsparameters.

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LIST OF SYMBOLS

Symbol	Definition	Units	Dimension
A, B	constants in eq.2.20	$kg.ha^{-1}.mm^{-1}.mbar^{-1}$	$L^{-2}T^{2}$
A	flow area		L^{2}
A	maximum water use efficiency	$kg.ha^{-1}.mm^{-1}.mbar^{-1}$	$L^{-2}T^{2}$
B	width at the waterline	m	L
B_{i}	total surface area of the catchment area of node i, open water excluded	m^{2}	L^{2}
B_{w}	Wet perimeter	m	L
C	differential moisture capacity	cm^{-1}	L^{-1}
Ch	Chezy friction coefficient	$m^{3}.s^{-2}$	$L^{3}T^{-2}$
$C_{b,i}$	net present value of the cost for each branch i	Dfl	-
C_{s}	net present value of the costs of the whole system	Dfl	-
$C_{s,g}$	net present value of the general costs for the whole system	Dfl	-
c	resistance for vertical flow *of poorly permeable layer*	d	T
$c_{E,i}$	yearly energy costs for branch i	Dfl	-
$c_{I,i}$	investment cost for branch i	Dfl	-
$c_{I,n,i}$	investment cost for branch i	Dfl	-
$c_{I,o,i}$	investment cost for branch i with the actual dimensions	Dfl	-
$c_{M,i}$	yearly maintenance costs for branch i	Dfl	-
$c_{s,g}$	yearly general costs for whole system for the Waterboard	Dfl	-
D	average thickness of the aquifer	m	L
D_{s}	development stage	-	-
$D_{s,c}$	development stage at full canopy	-	-
$D_{s,m}$	development stage at the start	-	

Symbol	Definition	Units	Dimension
	of maturing	-	-
$D_{s,t}$	development stage at tuber initation	-	-
$D_{s,e}$	development stage at the end of the early vegetative period	-	-
$D_{s,r}$	development stage used for simulation of root growth	-	-
D_O	thickness of the aquifer below the water level in the ditch	m	L
d_t	base flow at time t	$mm.s^{-1}$	
E	evapotranspiration rate	$kg.m^{-2}.s^{-1}$ $(mm.d^{-1})$	$ML^{-2}T^{-1}$
E_o	open water evaporation rate	$kg.m^{-2}.s^{-1}$ $(mm.d^{-1})$	$ML^{-2}T^{-1}$
E_p	potential evapotranspiration rate	$kg.m^{-2}.s^{-1}$ $(mm.d^{-1})$	$ML^{-2}T^{-1}$
E_s	soil evaporation rate	$kg.m^{-2}.s^{-1}$ $(mm.d^{-1})$	$ML^{-2}T^{-1}$
E_{sp}	potential soil evaporation rate	$kg.m^{-2}.s^{-1}$ $(mm.d^{-1})$	$ML^{-2}T^{-1}$
E_t	transpiration rate	$kg.m^{-2}.s^{-1}$ $(mm.d^{-1})$	$ML^{-2}T^{-1}$
E_{tp}	potential transpiration rate	$kg.m^{-2}.s^{-1}$ $(mm.d^{-1})$	$ML^{-2}T^{-1}$
F	open water area of the node	m^2	L^2
$F_{i,t}$	total open water area of node i at time t	m^2	L^2
Fr	Froude number	-	-
g	acceleration due to gravity $(g = 9.813)$	$m.s^{-2}$	MT^{-2}
h	water level	m	L
h_o	open water level	m	L
h_1,h_2	water level at the upstream (1) or downstream (2) end of branch	m	L
h_p	soil water pressure head	m	L
h_a	height of the phreatic surface averaged over the area	m	L
$h_{f,m}$	height of the phreatic surface midway between the watercourses	m	L
h_d	hydraulic head of deep aquifer	m	L
I	bottom slope	-	-
I_L	leaf area index	-	-

Symbol	Definition	Units	Dimension
i	internal rate of return	1	-
K	hydraulic conductivity	$m.d^{-1}$	LT^{-1}
K_M	roughness coefficient of Manning	$m^{1/3}.s^{-1}$	$L^{1/3}T^{-1}$
L	length of the branch	m	L
L_d	distance between watercourses	m	L
L_r	depth of root zone	m	L
$L_{r,c}$	depth of the root zone at full canopy	m	L
$L_{r,m}$	depth of the root zone at the start of maturing	m	L
$L_{r,h}$	depth of the root zone at harvest	m	L
l_c	crop height	m	L
$l_{c,c}$	crop height at full canopy	m	L
$l_{c,m}$	crop height at start maturing	m	L
$l_{c,h}$	crop height just before harvest	m	L
N	lifetime of project	a	T
n	total number of branches		-
P	gross rainfall	$mm.d^{-1}$	LT^{-1}
P_t	gross rainfall at time t	mm	L
P_a	atmospheric pressure $(p_a = 1013)$	mbar	$ML^{-1}T^{-2}$
P_c	gross growth rate for clear days	$kg.ha^{-1}.d^{-1}$	$ML^{-2}T^{-1}$
P_o	gross growth rate for overcast days	$kg.ha^{-1}.d^{-1}$	$ML^{-2}T^{-1}$
P_{st}	gross potential growth rate	$kg.ha^{-1}.d^{-1}$	$ML^{-2}T^{-1}$
$p_{p,i}$	percentage paved area of F_i	-	-
$p_{u,i}$	percentage unpaved area of F_i	-	-
Q_{act}	total actual yield	$kg.ha^{-1}$	ML^{-2}
$Q^j_{act,i}$	total actual yield for variant j in year i	$kg.ha^{-1}$	ML^{-2}
$Q^j_{dep,i}$	yield depression for variant j in year i	-	-
$Q_{e,i,t}$	total lateral inflow towards node i at time t	$m^3.s^{-1}$	L^3T^{-1}
Q_{pot}	total potential yield	$kg.ha^{-1}$	ML^{-2}
$Q^j_{pot,i}$	total potential yield for variant j in year i	$kg.ha^{-1}$	ML^{-2}

Symbol	Definition	Units	Dimension
$Q^j_{rel,i}$	total relative yield for variant j in year i	-	-
$\dot{q}$	growth rate	$kg.ha^{-1}.d^{-1}$	$ML^{-2}T^{-1}$
q	extraction rate of water	$m.d^{-1}$	LT^{-1}
q_a	flow to/from the deep aquifer	$mm.d^{-1}$	LT^{-1}
q_d	flow to/from ditches/drains	$mm.d^{-1}$	LT^{-1}
q_D	drained flux from the system	$mm.d^{-1}$	LT^{-1}
q_m	vertical flux per unit area midway between the ditches	$mm.d^{-1}$	LT^{-1}
$\dot{q}_{act}$	actual production rate under limiting conditions	$kg.ha^{-1}.d^{-1}$	$ML^{-2}T^{-1}$
$\dot{q}_{max}$	maximum production rate under limiting conditions	$kg.ha^{-1}.d^{-1}$	$ML^{-2}T^{-1}$
$\dot{q}_{pot}$	potential production value under limiting conditions	$kg.ha^{-1}.d^{-1}$	$ML^{-2}T^{-1}$
q_S	supplied flux to the system	$mm.d^{-1}$	LT^{-1}
$q_{p,i,t}$	specific discharge resulting from rainfall on paved area in the catchment area of node i at time t	$m.s^{-1}$	LT^{-1}
$q_{p,i,t}$	specific discharge resulting from rainfall on unpaved area in the catchment area of node i at time t	$m.s^{-1}$	LT^{-1}
r	rate of discount	-	-
R	hydraulic radius	m	L
S	sink term/root water uptake	d^{-1}	T^{-1}
S_{max}	maximum possible root water uptake	d^{-1}	T^{-1}
S_c	fraction of soil covered by crop	-	-
$S_{c,h}$	fraction of soil covered by crop at harvest	-	-
t	time parameter		T
t_e	emergence date	-	-
t_h	harvest date	-	-

Symbol	Definition	Units	Dimension
w	radial resistance	$d.m^{-1}$	$L^{-1}T$
z	vertical coordinate with origin at surface, positive direction upward	cm	L
α	reaction factor	d^{-1}	T^{-1}
α_S	reduction coefficient for S_{max}	1	-
β	ratio of harvested part over total production	-	-
β_e	ratio of harvested part over total production at the end of the early vegetative period	-	-
β_h	ratio of harvested part over total production at harvest	-	-
Δe	saturation vapour pressure deficit	mbar	$ML^{-1}T^{-2}$
Δt	length of time step		T
η_i	coefficient for the shape of the phreatic surface	-	-
Θ	time centre coefficient	-	-
ξ	a mathematical flexibility constant close to zero.	-	-
T	drainage resistance	d	T

50

80